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Verification of the Water Quality Impacts of Combined Sewer Overflow

Metropolitan Sewerage District of the County of
Milwaukee, WI

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16. ABSTRACT

The purpose of this study was to identify the source and mechanism of the water quality impacts in the Milwaukee River following wet weather discharges. Monitoring surveys have demonstrated that there is often a rapid decline in DO following CSO discharge events in the lower portions of the river. Water quality modeling of the river with Harper's water quality model indicate that the loadings from combined and storm sewer discharges are not sufficient to cause the observed rapid declines in DO. The results of laboratory and field investigations indicate that bottom sediments within the lower river are a significant sink for DO and are linked to the rapid loss of DO. The mechanism of this rapid DO decline is the scouring of sediment oxygen demanding materials by submerged CSO outfalls. This was determined through measurements of instream velocities near the bottom sediments resulting from discharges from submerged outfalls. Empirical equations were developed using multiple regression analysis to predict the impact of sediment scouring by CSO discharges on DO levels. An expression was also added to Harper's water quality model to provide for time varying SOD rates which are required to simulate the high oxygen demand of scoured sediments. This model was calibrated and verified for dry and wet weather conditions in the study area of the river and was used to determine the DO and fecal coliform impact which is attributable to CSO.

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FOREWORD

The Environmental Protection Agency was created because of increasing public and government concern about the dangers of pollution to the health and welfare of the American people. Noxious air, foul water and spoiled land are tragic testimony to the deterioration of our natural environment. The complexity of that environment and the interplay between its components require a concentrated and integrated attack on the problem.

Research and development is that necessary first step in problem solution and it involves defining the problem, measuring its impact and searching for solutions. The Municipal Environmental Research Laboratory develops new and improved technology and systems for the prevention, treatment and management of wastewater and solid and hazardous waste pollutant discharges from municipal and community sources, for the preservation and treatment of public drinking water supplies, and to minimize the adverse economic, social health and aesthetic effects of pollution. This publication is one of the products of that research; a most vital communication link between the researcher and the user community.

This report describes water quality impacts associated with wet weather discharges into the Milwaukee River and details the contribution of combined sewer overflows to this impact. Through this project data are being obtained to determine in a rational way the degree of national wet weather pollution control required.

Francis T. Mayo, Director
Municipal Environmental
Research Laboratory

ABSTRACT

The purpose of this study was to identify the source and mechanism of the water quality impacts in the Milwaukee River following wet weather discharges. The results of intensive field surveys were used to identify the river's response in terms of dissolved oxygen (DO) and fecal coliform concentrations to different rainfall events under a range of river flow conditions. River dye studies indicate that velocities are extremely slow even during high flow conditions in the lower portions of the Milwaukee River which are under the influence of Lake Michigan. As a result of these slow velocities, sediments accumulate in the lower river. Laboratory and field investigations indicate that these bottom sediments are a significant sink for DO. Studies of the in situ sediment oxygen demand (SOD) rates using respirometers and chemical analyses of sediment core samples were utilized to develop sediments maps illustrating the magnitude and distribution of these parameters. SOD rates ranged from 1.8 to 6.7 gm O_2/m^2-day in the lower river. Bench scale tests conducted with bottom sediments show that the oxygen demand of disturbed or agitated sediments exceeded 1000 gm O_2/m^2-day at some locations. This is more than 100 times greater than the SOD rates measured in situ.

Continuous monitoring of DO and temperature at several locations in the lower Milwaukee River and the results of the intensive monitoring surveys have demonstrated that there is often a rapid decline in DO following combined sewer overflow (CSO) events. Water quality modeling of the river with Harper's water quality model and modeling results of other investigators indicate that the loadings from combined and storm sewer discharges are not sufficient to cause the observed rapid declines in DO. The mechanism of this rapid decline is the scouring of sediment oxygen demanding materials by submerged CSO outfalls. This was determined through measurements of instream velocities near the bottom sediments resulting from discharges from submerged outfalls and settling tests of sediments which indicate the velocity required to scour sediments. Empirical equations were developed using multiple regression analysis to predict the impact of sediment scouring by CSO discharges on DO levels. An expression was also added to Harper's water quality model to provide for time varying SOD rates which are required to simulate the high oxygen demand of scoured sediments. This model was calibrated and verified for dry and wet weather conditions in the study area of the river and was used to determine the DO and fecal coliform impact which is attributable to CSO.

This report was submitted in partial fulfillment of research grant no. EPA R804518 by the Metropolitan Sewerage District of the County of Milwaukee under the sponsorship of the U.S. Environmental Protection Agency. This report covers the period of August 15, 1976 to December 31, 1978 and was completed December 31, 1978.

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SECTION I

CONCLUSIONS

1. Sediment oxygen demand (SOD) is the major source of the dissolved oxygen (DO) impacts in the lower Milwaukee River and has its greatest effect during low river flow conditions.

2. The mechanism of the DO depletion observed in the Milwaukee River during wet weather is the scouring and resuspension of the bottom sediments by submerged combined sewer overflow outfalls.

3. The oxygen demand of disturbed sediments can be greater than 1000 gm/m^2-day which is more than 100 times greater than the demand of undisturbed sediments.

4. Combined sewer overflows (CSO) contribute approximately 40 to 50 percent of the annual loadings of oxygen demanding materials (in terms of carbonaceous BOD) and suspended solids to the sediments in the lower Milwaukee River.

5. The relationship of rainfall, runoff and prestorm history to the variable sediment oxygen demand can be predicted through the use of a statistical procedure.

6. The major source of fecal coliforms in the Milwaukee River is combined sewer overflow and removal of these overflows will nearly eliminate fecal coliform standards violations.

SECTION 2

RECOMMENDATIONS

1. The significance of sediment oxygen demand as a sink for dissolved oxygen in receiving waters should be investigated in other areas of the United States.
2. Investigate the feasibility of the periodic implementation of a dredging program in the Milwaukee River as a means of alleviating dissolved oxygen problems.

SECTION 3

INTRODUCTION

The presence of combined sewer systems in the United States and Canada has been extensively documented in numerous literature sources (1,2). The effects of discharges from these systems on the receiving waters has become increasingly important to municipalities and regulating agencies because of the directives of PL 92-500, the Federal Water Pollution Control Act Amendments of 1972, which states:

> "Wherever attainable, an interim goal of water quality which provides for the protection and propagation of fish, shellfish, and wildlife and provides for recreation in and on the water shall be achieved by July 1, 1983". (Section 101a)

Significant amounts of data have been generated in studies of the quantity and quality of combined sewer overflow (CSO) from locations throughout North America (3,4). In addition, demonstration studies at numerous locations have been completed which provide data on treatment or other control measures for reducing the discharge loads from CSO areas (5-8). More recently, comprehensive projects have been proposed, or have already initiated major expenditures of funds for the abatement of CSO. Typical of major projects are the following cities efforts:

Chicago, IL - $1.8 billion program for CSO and related flood control (9).
Rochester, NY - $0.4 billion to abate CSO (9).
Milwaukee, WI - $1.5 billion for the control of CSO and upgrading of treatment facilities (10).

Each of these projects and countless more in other metropolitan areas will be required to identify in various levels of detail, the improvements in water quality as a function of dollars for CSO control (EPA PRM 75-34). In order to develop examples of the magnitude of the water quality impacts associated with CSO, EPA has conducted or is in the process of conducting site specific water quality studies at a few locations. This study, "Verification of Water Quality Impact from CSO" is one of these projects whose objectives are two-fold. The first objective was to use real-time data collected through monitoring surveys to characterize the water quality impacts associated with wet weather discharges. The monitoring surveys included continuous data collection, intensive grab sample surveys lasting 3 or 4 days and special studies to establish the mechanism of the DO impacts. The second objective was to decipher the contribution of CSO to this impact so that a general relationship can be generated for the study area.

PROJECT STUDY AREA

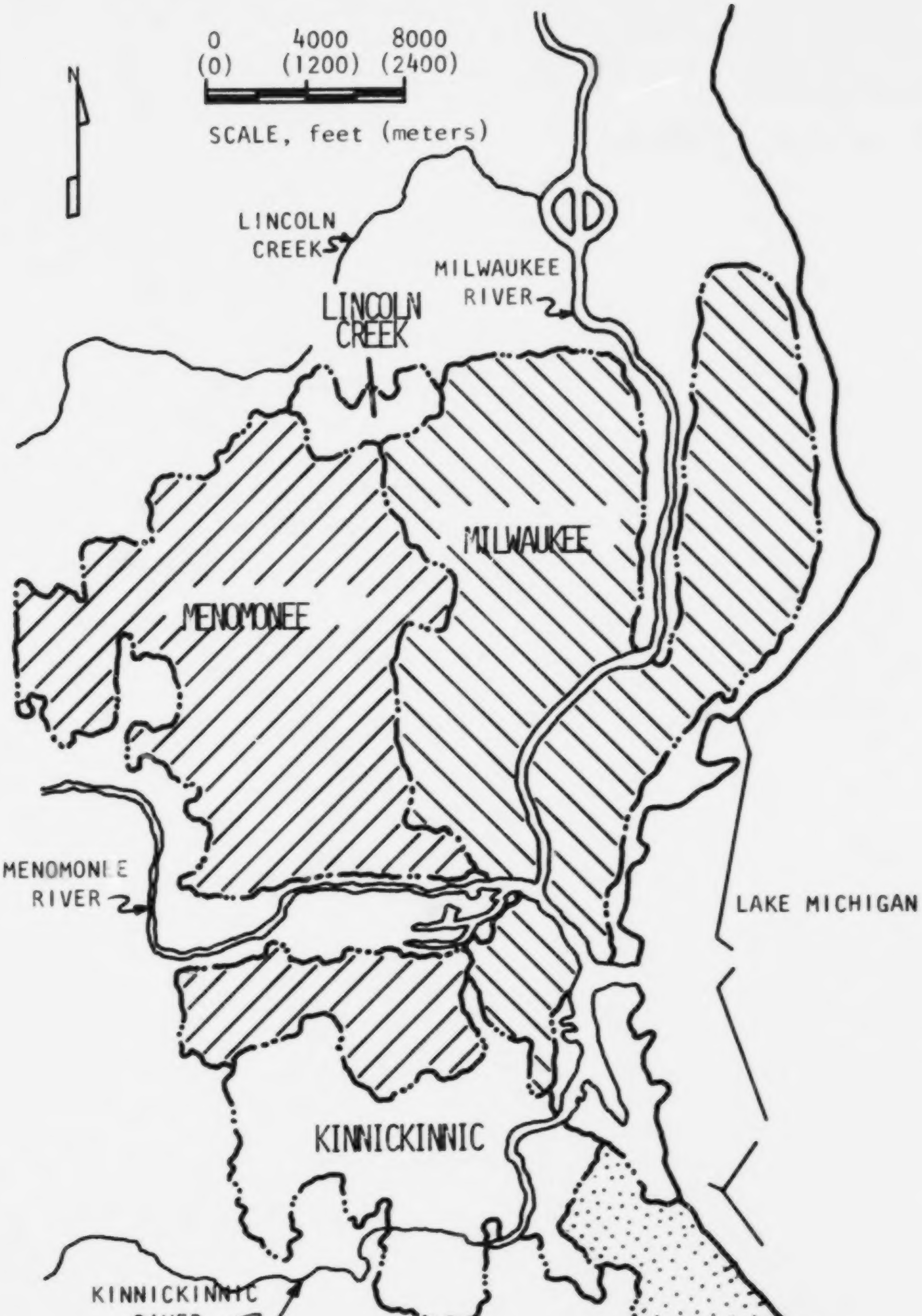
The Milwaukee area was selected as one of the sites for the impact evaluation project because of its large combined sewer area, the availability of previous and ongoing projects relating to CSO, and the significant water quality impacts which occur as a result of wet weather discharges. A map of the three rivers that enter Lake Michigan at the heart of the metropolitan Milwaukee area is presented in Figure 1. Each of the rivers has an associated combined and storm sewer area which discharges during wet weather. For purposes of this study, the investigations into the water quality impacts were limited to the Milwaukee River for the following reasons:

1. It is the largest of the three rivers and it contains the largest contribution of CSO.
2. It has a definite physical boundary which separates the lower reach of the river into sections influenced and not influenced by Lake Michigan.
3. Previous projects related to the CSO problem have been undertaken on this river.
4. The other two rivers have similar downstream impact characteristics which would provide no unique data to the analysis.

Milwaukee River

The drainage area characteristics of the Milwaukee River watershed are listed in Table 1. Figure 2 presents the location of the river monitoring points and other important features of the Milwaukee River study area. The most dominant feature of the river is the inflow from Lake Michigan to the lower reaches of the river which influences flow conditions as far upstream as the North Avenue Dam (Figure 3). The inflow of lake water results in the slowing or reversal of flow in these reaches throughout the year. The lake influence has been monitored at a few downstream locations with a sensitive current speed and direction meter. The findings of these surveys have shown the river to be flowing upstream at one depth in the water column and downstream at another depth with a complete reversal of this trend a few moments later. Attempts to relate the inflows to wind speed, river flow and lake level have proved fruitless.

The mean annual discharge for the river as measured by the USGS gauge at Estabrook Park is 400 cfs (11.3 m^3/sec) with the monthly variation in flow shown in Figure 4. During the coldest portion of the winter months, the river is ice covered in both the lower and upper reaches. The CSO area that discharges to the river extends along both sides of the river with the first outfall located at Capitol Drive as shown in Figure 2. A majority of the outfalls in the lower portions of the river are completely or partially submerged. Typical cross-sections of the river at upstream and downstream locations and photographs of these sites are shown in Figures 5 through 8.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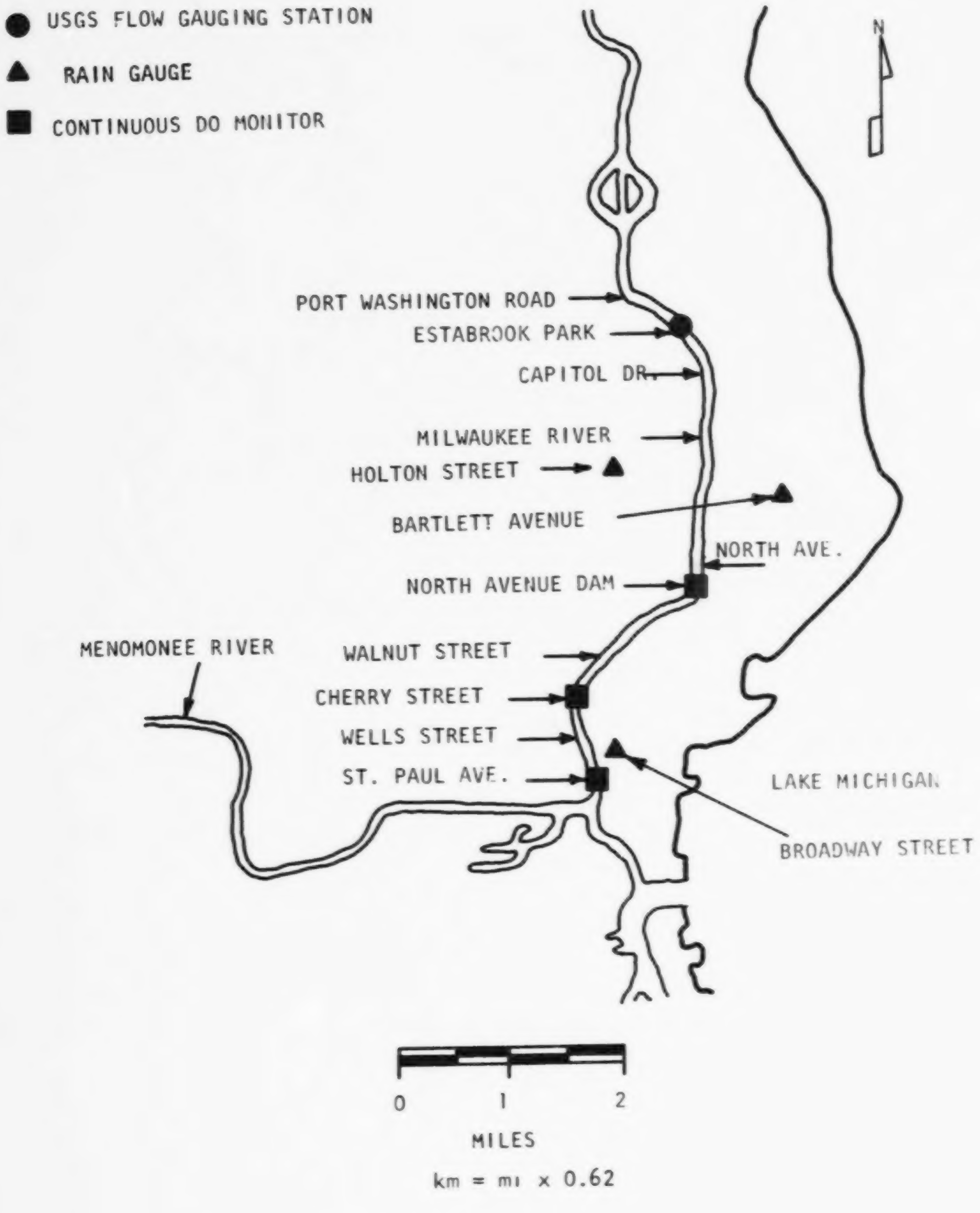
LAKE MICHIGAN

Figure 1. Illustration of the CSO drainage areas contributing to the various receiving waters in Milwaukee, Wisconsin.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▲ RAIN GAUGE
■ CONTINUOUS DO MONITOR
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CAPITOL DR.
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HOLTON STREET
BARTLETT AVENUE
NORTH AVE.
NORTH AVENUE DAM
MENOMONEE RIVER
WALNUT STREET
CHERRY STREET
WELLS STREET
ST. PAUL AVE.
LAKE MICHIGAN
BROADWAY STREET
0
1
2
MILES
km = mi x 0.62

Figure 2. Monitoring locations in the study area of the Milwaukee River.

TABLE 1. MILWAUKEE RIVER WATERSHED CHARACTERISTICS

Total drainage area - mi^2 (km^2)	695 (1800)
Population	530,000
Land use - percent of area	
Residential	6.1
Commercial	0.3
Industrial	0.4
Agricultural	61.0
Open	32.2
Combined sewer drainage area - mi^2 (km^2)	9.5 (24.6)
Estabrook Park flow - cfs (m^3/sec)	
Mean annual	400.0 (11.3)
Seven day - ten year low flow	19.2 (0.5)
100 year flood flow	14000.0 (396.5)

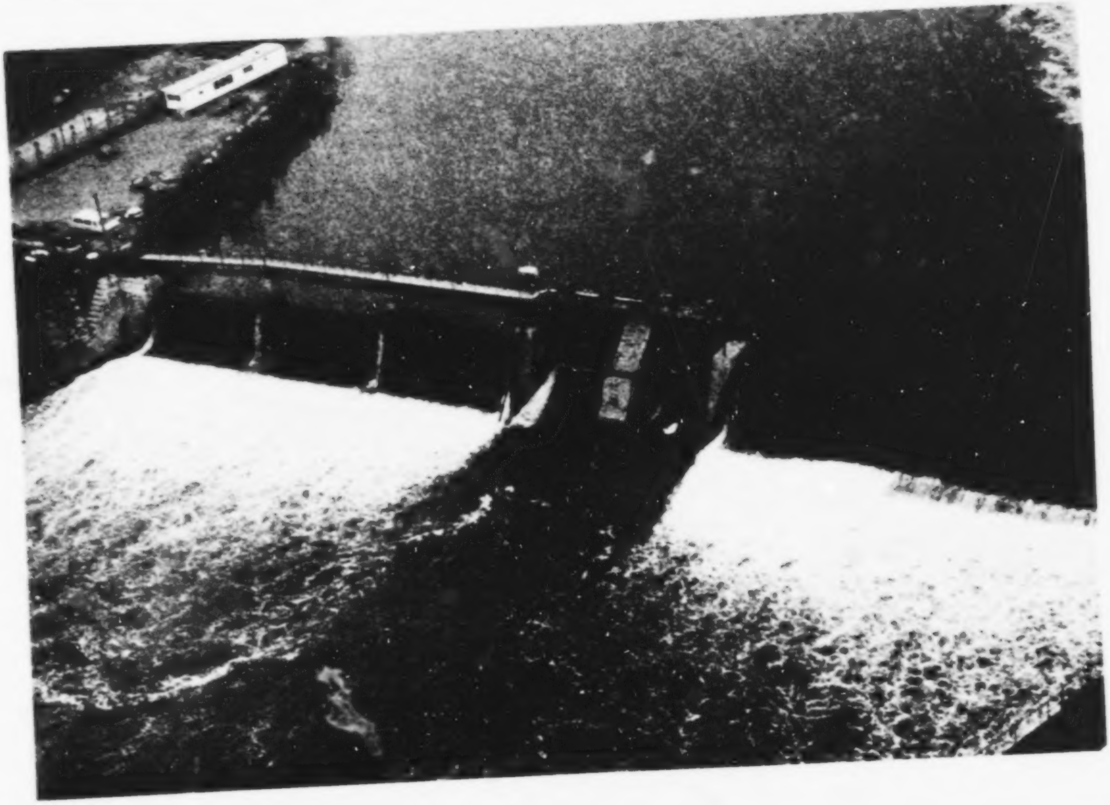


Figure 3. Photograph of the North Avenue Dam on the Milwaukee River.

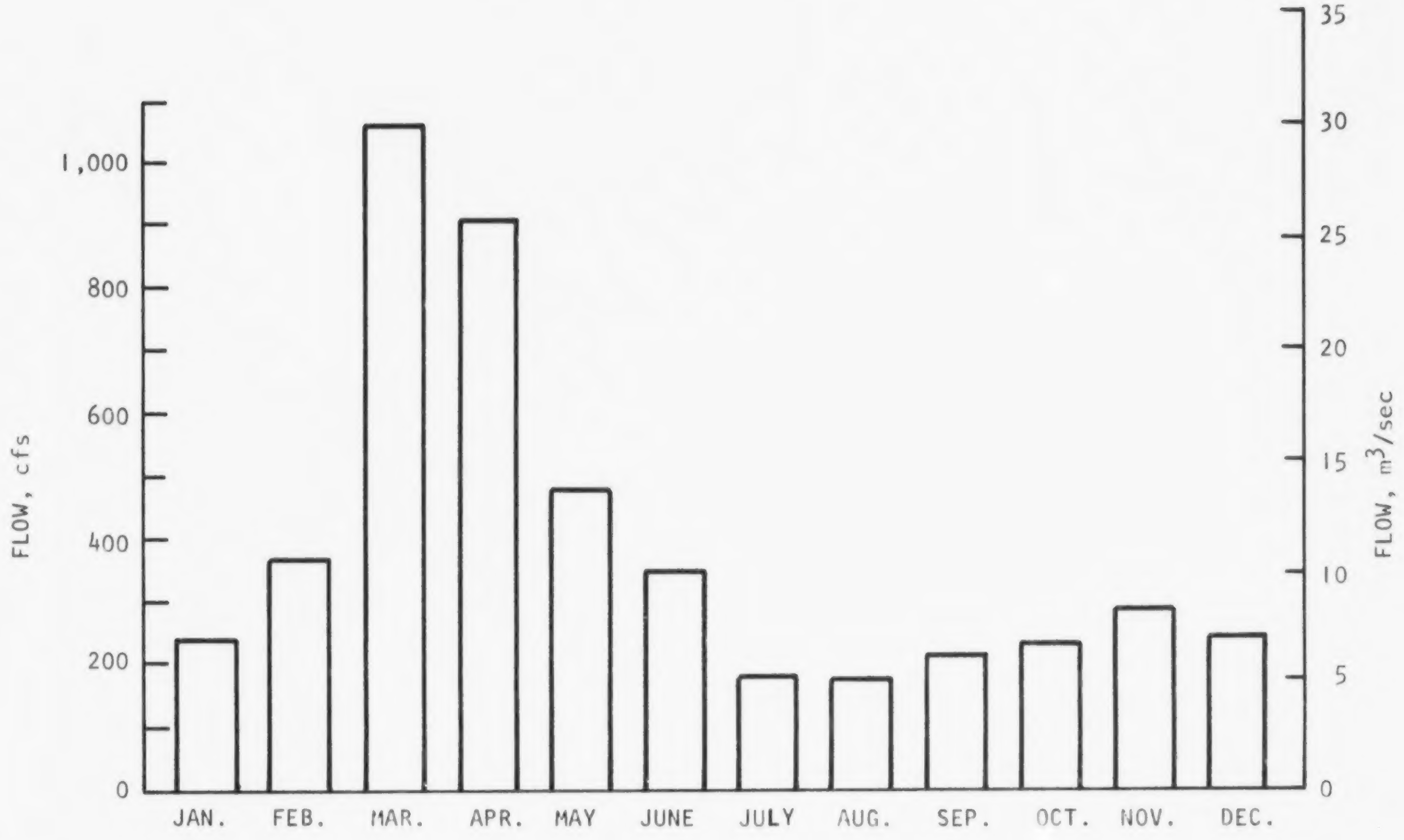
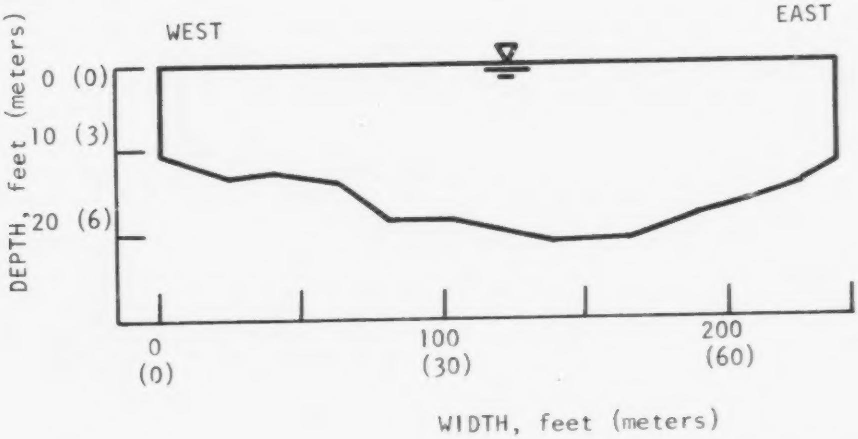


Figure 4. Mean monthly flows for the Milwaukee River at the Estabrook Park USGS flow gauging station.



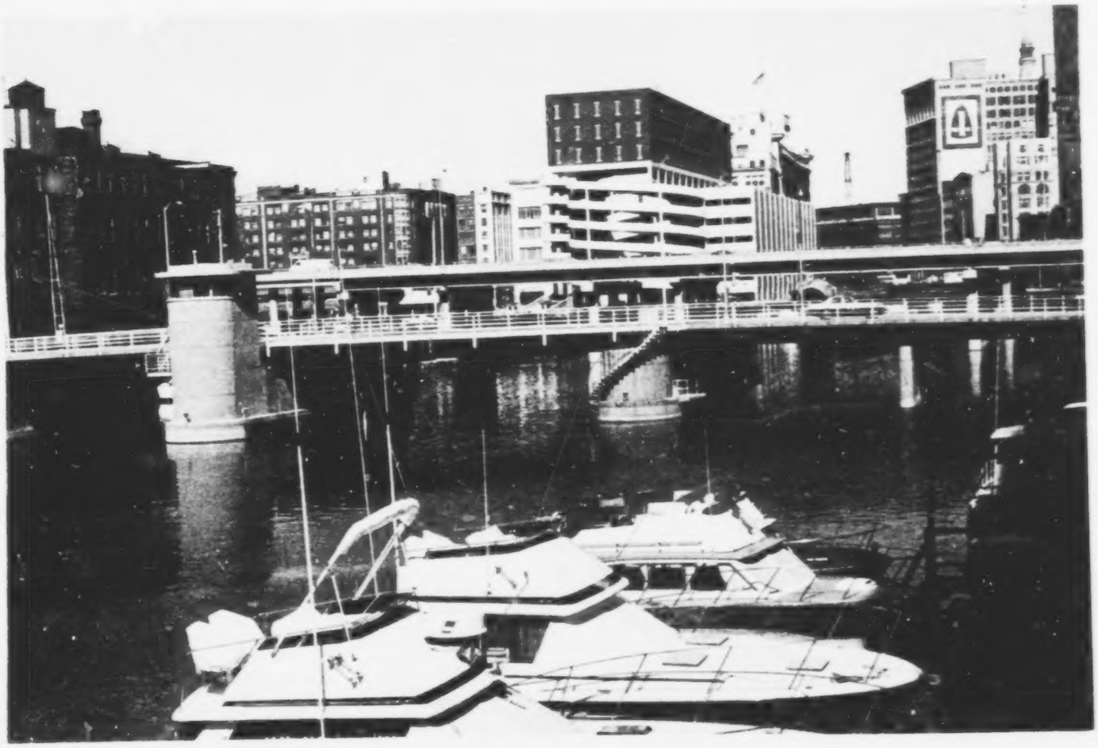


Figure 5. Photograph and cross section of the Milwaukee River at St. Paul Avenue.

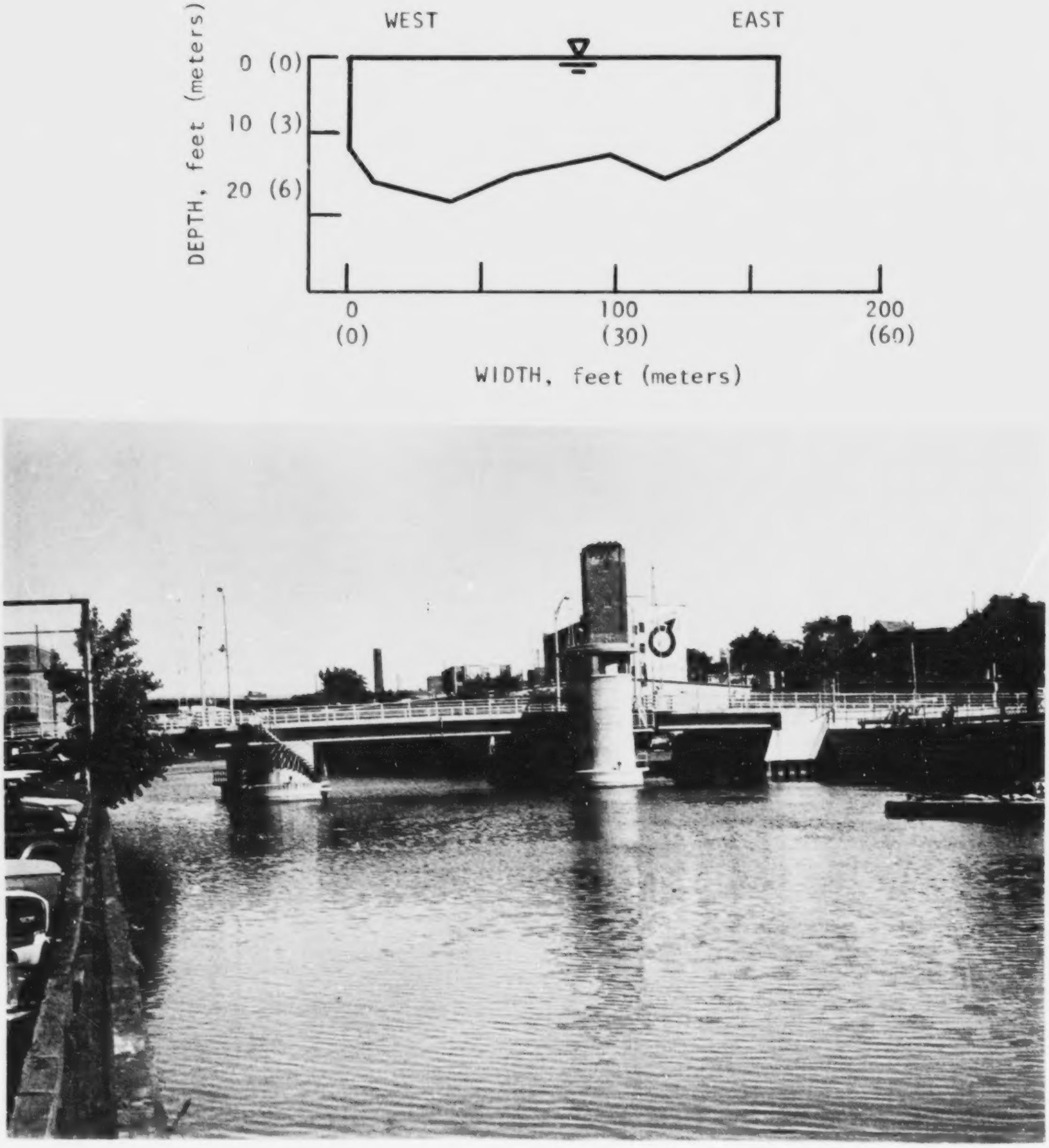
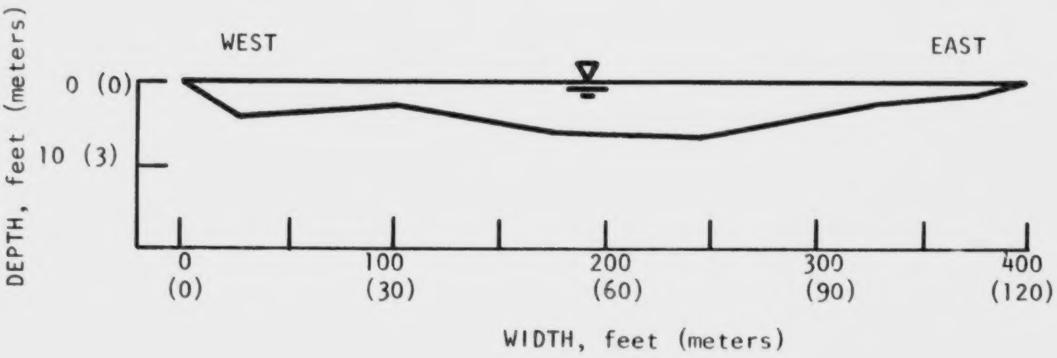


Figure 6. Photograph and cross section of the Milwaukee River at Walnut Street.



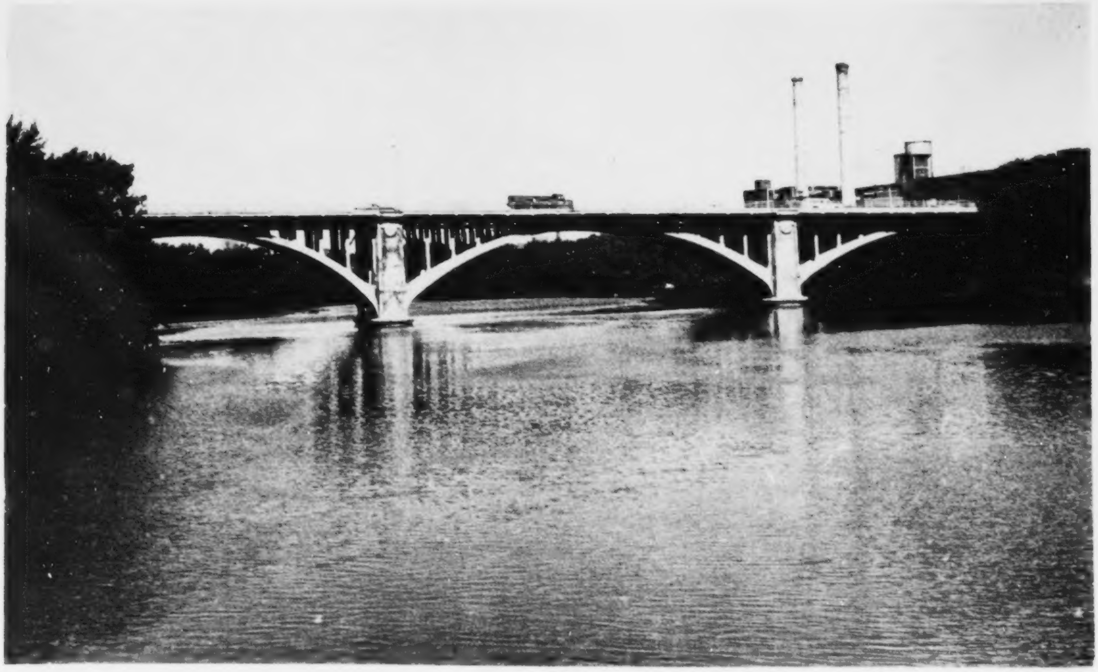


Figure 7. Photograph and cross section of the Milwaukee River at North Avenue.

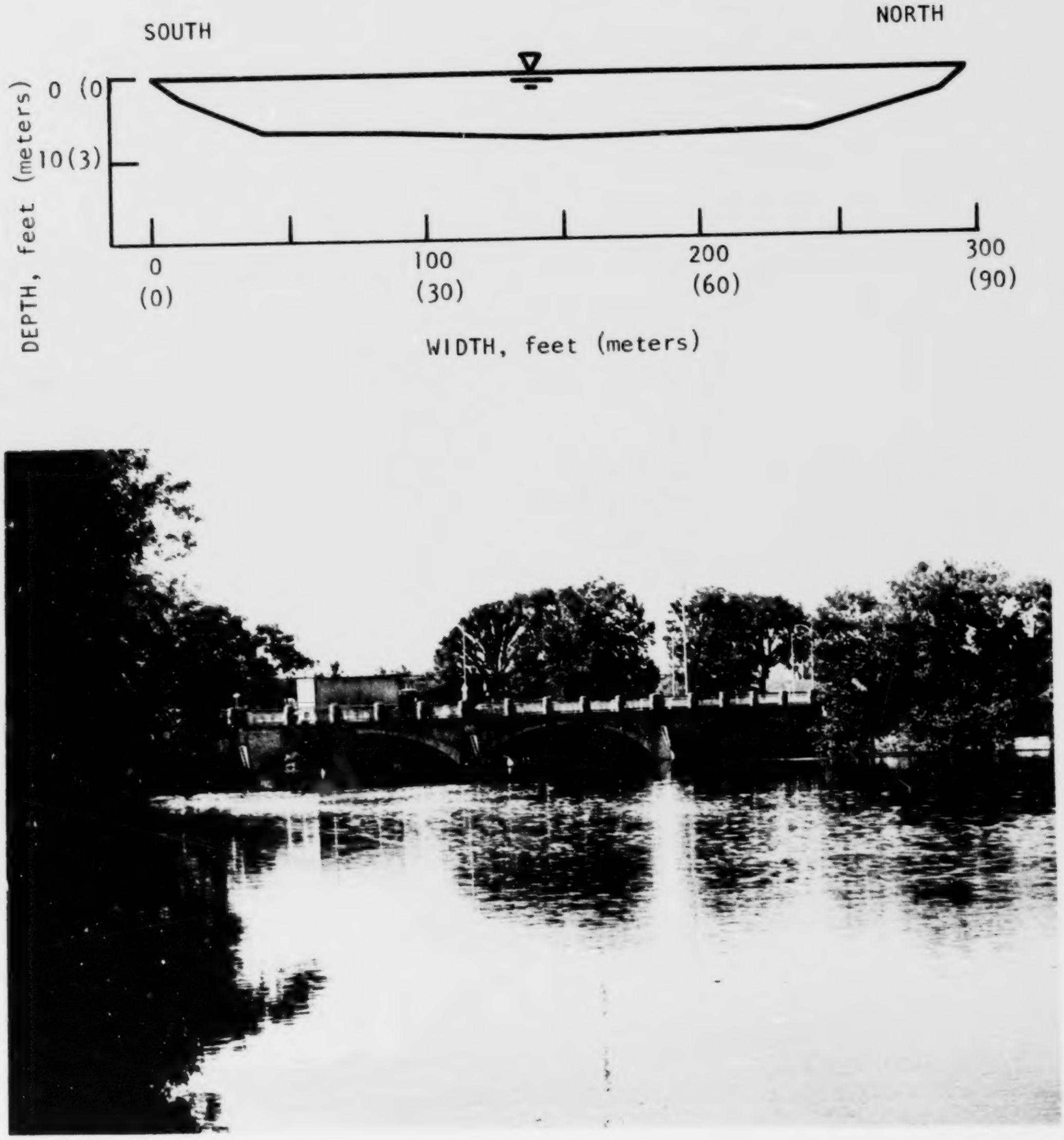

SOUTH
NORTH
0 (0)
10(3)
DEPTH, feet (meters)
0
(0)
100
(30)
200
(60)
300
(90)
WIDTH, feet (meters)

Figure 8. Photograph and cross section of the Milwaukee River at Port Washington Road.

Combined Sewer Area

The combined sewer area that contributes to the Milwaukee River is approximatly 6000 ac (2428 ha) with a total of 52 outfalls ranging in size from 12 in. (30.5 cm) to a double 10 by 7.5 ft (3 by 2.3 m) box outfall. The drainage areas for individual outfalls range in size from 5 to 702 ac (2 to 284 ha). The entire combined sewer area has been modeled in a previous project using the EPA Storm Water Management Model (SWMM) and the Army Corps of Engineers Storage Treatment Overflow Runoff Model (STORM)(11). The land use percentages of the CSO area from the model data are as follows:

Land use	Percent of CSO area
Single family residential	57.6
Multi family residential	12.6
Commercial	17.8
Industrial	11.3
Parkland	0.7

Figure 9 presents a representation of the CSO drainage area that contributes to the river as a function of distance from Lake Michigan.

Storm Sewer Area

Those areas within Milwaukee County that drain to the Milwaukee River and are served by storm sewers comprise approximately 27,000 ac (10,900 ha) of drainage area. These areas contain the following land use breakdown:

Land use	Percent of storm sewer area
Single family residential	50.1
Multi family residential	2.7
Commercial	12.9
Industrial	4.4
Parkland - open area	29.9

This area has also been modeled with the SWMM and STORM models in a previous project (11). A large portion of this drainage area contributes to Lincoln Creek (Figure 1) which eventually discharges to the Milwaukee River. Figure 10 illustrates a comparison of the storm sewer and CSO areas. Some of the storm sewers within this area contain cross connections with the sanitary sewers so that during wet weather the surcharged sanitary system may be relieved through nearby storm sewers. Further descriptions of this system will be developed in Section 6, Modeling Studies.

Upstream Areas

The Milwaukee River watershed is comprised of 644 mi^2 (1668 km^2) of drainage area that lies to the north of the Milwaukee metro area. The predominant land use within this area is agricultural in nature which comprises approximately 66 percent of the upstream watershed. Urban land use accounts for 1.5 percent while open areas and woodlands make up the remaining area. In 1970, an estimated 530,000 persons resided in the entire watershed including the area within Milwaukee County.

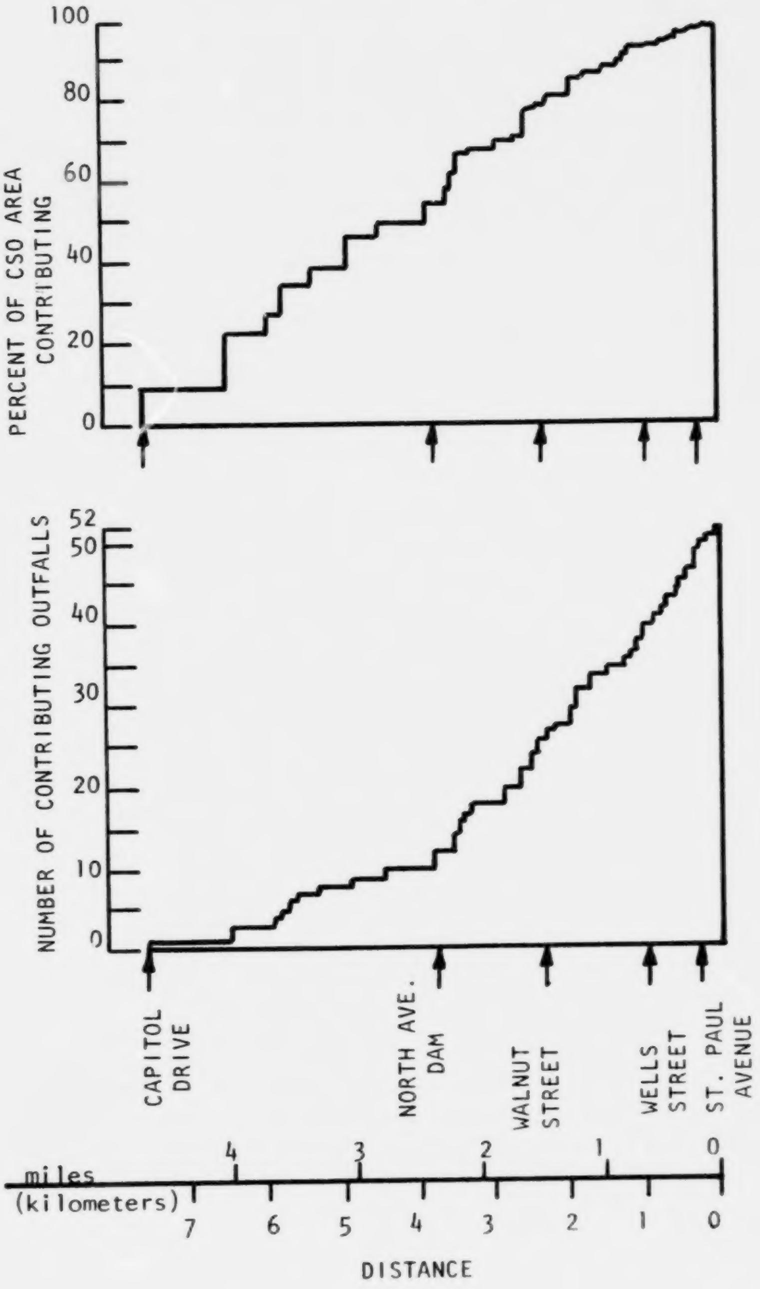


Figure 9. Illustration of the distribution of CSO outfalls and contribution from the CSO drainage area along the Milwaukee River.

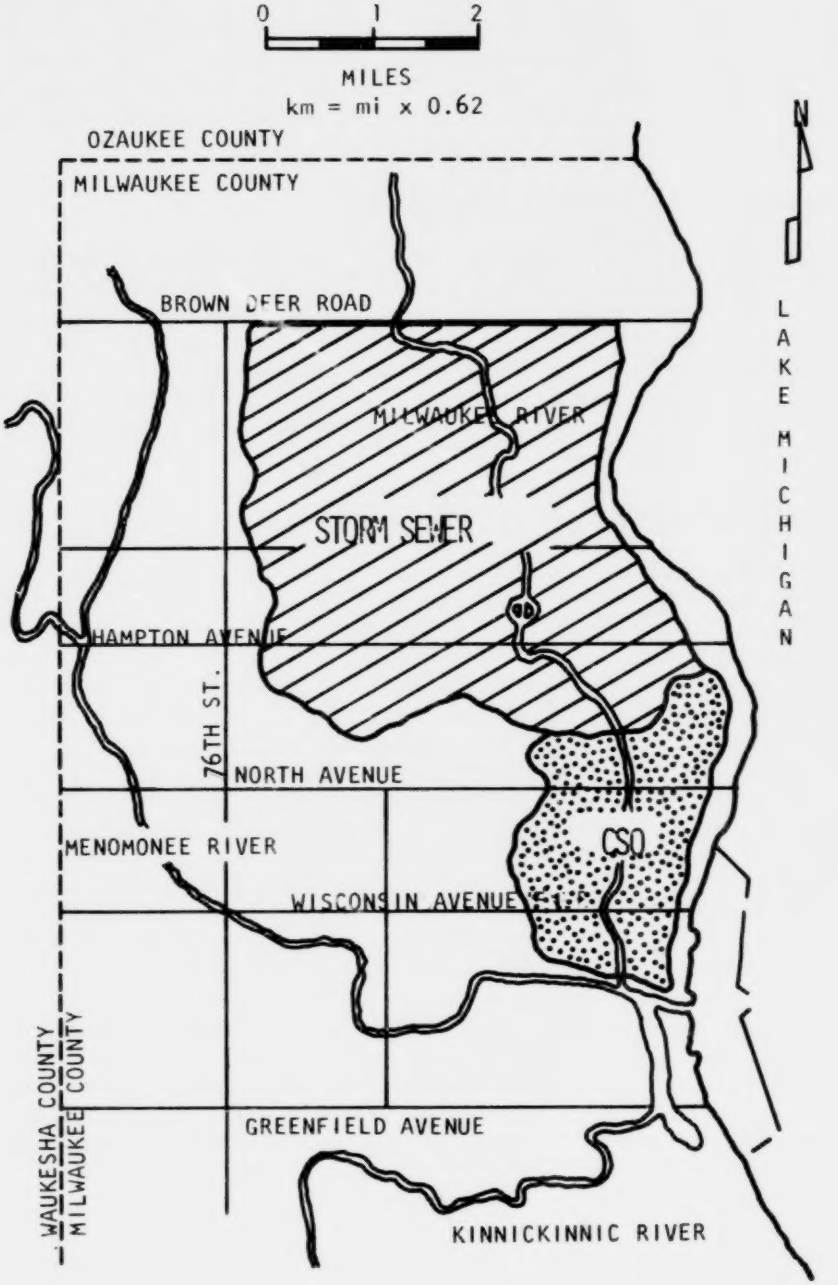

0
1
2
MILES
km = mi x 0.62
OZAUKEE COUNTY
MILWAUKEE COUNTY
N
BROWN DEER ROAD
MILWAUKEE RIVER
STORM SEWER
HAMPTON AVENUE
76TH ST.
NORTH AVENUE
CSO
MENOMONEE RIVER
WISCONSIN AVENUE
GREENFIELD AVENUE
KINNICKINNIC RIVER
WAUKESHA COUNTY
MILWAUKEE COUNTY
LAKE MICHIGAN

Figure 10. Illustration of CSO and storm sewer areas tributary to the Milwaukee River.

PROJECT BACKGROUND

Two projects within the Milwaukee area that have dealt with some aspect of the CSO problem have preceded the subject project. Some of the findings and data from these projects were utilized in the subject project to verify these investigations. In order to acquaint the reader with this background history, the following brief discussions are presented.

Humboldt Avenue Detention Tank Project (12)

Within this demonstration project, the City of Milwaukee evaluated the merits of detention tanks as a practical method for the abatement of CSO. A 3.9 million gal. (14,800 m^3) CSO detention tank was constructed to intercept overflow from a 570 ac (230.8 ha) segment of the Milwaukee River CSO area. As part of the evaluation of the facility, an extensive sewer and river monitoring program was conducted in conjunction with a detention tank and river modeling program. This project was carried out under partial sponsorship of EPA and was completed in late 1972.

Milwaukee Combined Sewer Overflow Pollution Abatement Project (11)

The Milwaukee Metropolitan Sewerage District in late 1974 initiated a study of the entire CSO area within Milwaukee to assess the impact of CSO on the area rivers and to develop the most cost effective CSO abatement alternative to improve water quality. This project was partially funded by EPA as a Section 201 Construction grant with the firm of Stevens, Thompson and Runyan (now STRAAM Engineers) as the prime consultant. The 3.25 million dollar project included water quality monitoring and modeling to define the frequency and magnitude of any instream impacts. The modeling of the entire CSO area using the SWMM and STORM models was completed in this project as well as the instream modeling of the Milwaukee River using Harper's water quality model. The water quality evaluations of this project were limited to the determination of impacts associated with various levels of protection for selected abatement alternatives using design storm analyses. The project is presently evaluating the abatement strategy of conveyance-storage-treatment using deep tunnel conveyance and mined underground storage.

An additional portion of the Milwaukee CSO project was the modeling requirements necessary to meet the PRM 75-34 requirements of EPA which states that the selection of the most cost effective CSO abatement alternative will be based upon a comparison of the relationship between the water quality improvement associated with each abatement measure and its resulting cost. For this project the Environmental Sciences Division of Envirex (now the Environmental Research Center of Rexnord Inc.) with assistance from Harper-Owes Consultants was contracted to carry out the above analyses (13). In order to evaluate the frequency and magnitude of water quality standards violations, Harper's model was modified to simulate long term periods (years) of rainfall. The results of this portion of the CSO project have shown the conveyance-storage-treatment concept to be the most effective in improving the water quality of the Milwaukee River when it is designed for a 1/2 year recurrence interval storm. Storms of lesser frequency provide little additional improvement for a substantial increase in cost.

REPORT ORGANIZATION

The remaining portions of this report will describe the data collection and analyses used to define the CSO impact within the Milwaukee area. Section 4, Field and Laboratory Investigations, will describe the monitoring tasks within the project that were used to quantify the magnitude of the CSO impacts during and after wet weather discharges. These investigations include the instream and combined sewer monitoring that was used to verify the model network used in the evaluation of the impacts. Section 5, Analysis of Data, uses the data of Section 4 to determine the source and mechanism of the wet weather impacts. Section 6, Modeling Studies, describes how the model network was used to simulate the water quality conditions which led to the evaluation of the CSO contribution to the individual impacts. Descriptions of the instream model are also provided in this section. Section 7, Evaluation of CSO Impact, contains the analysis of the CSO impact and relates the improvements in water quality associated with control techniques. The benefits of CSO control, as well as the relationship of this project to a national evaluation are also included.

SECTION 4

FIELD INVESTIGATIONS

The water quality conditions within the Milwaukee River exhibit extreme variations between wet and dry weather periods. Figure 11 presents data from a continuous DO monitoring device located at St. Paul Avenue showing the variation in DO. Figure 12 presents fecal coliform data from the same site during two intensive monitoring surveys. The dramatic loss of dissolved oxygen and rapid increases in fecal coliform concentrations were investigated in the field monitoring portions of this project. These consisted of of comprehensive field surveys, the analysis of two years of data taken from three continuous DO and temperature monitors (Figure 2), dye studies to define the rivers hydraulic and mixing characteristics, and sediment investigations.

This section of the report will describe these four components of the field monitoring program which were used to decipher the contribution of CSO to this impact. The term water quality impact for this study will be limited to the violations of the DO and fecal coliform standards that have been set by the Wisconsin Department of Natural Resources for the Milwaukee River. The standards apply for both dry and wet weather flow conditions in the river and are as follows:

Upstream of North Avenue Dam

1. Preservation and enhancement of fish and other aquatic life. The DO content shall not be lowered to less than 5 mg/l at any time.

2. Full body contact recreational use; the membrane filter fecal coliform count (MFFCC) shall not exceed 200 per 100 ml as a geometric mean based on not less than five samples per month.

Downstream of North Avenue Dam

1. Marginal conditions for fish and aquatic life. The DO shall not be lowered to less than 2 mg/l at any time.

2. Partial body contact recreational use; the MFFCC shall not exceed 1000 per 100 ml as a geometric mean based on not less than five samples per month.

The difference in standards between the upper and lower portions of the river reflect the physical and biological characteristics of these two

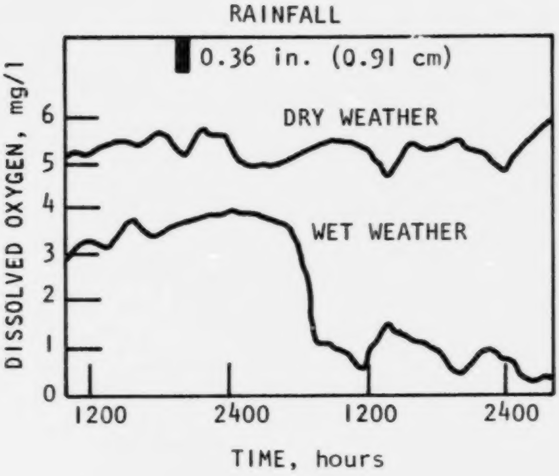


Figure 11. Illustration of dry weather (July 29-31, 1977) and wet weather (August 3-5, 1977) DO levels at St. Paul Avenue in the Milwaukee River.

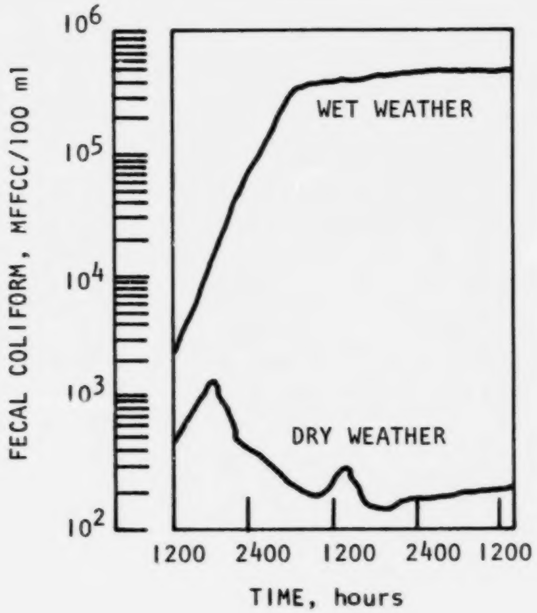


Figure 12. Illustration of dry weather (September 22-24, 1976) and wet weather (August 4-6, 1977) fecal coliforms levels at St. Paul Avenue in the Milwaukee River

reaches. The upper portions above the dam are free flowing and not under the influence of Lake Michigan while the lower portions are those reaches which exhibit some degree of inflow from the lake. Figure 9 presents the distribution of the CSO drainage area as a function of distance along the river. The North Avenue Dam is located where approximately 55 percent of the CSO area has contributed to the upstream portions of the river, but only 12 of the 52 outfalls along the river discharge upstream of this location.

The field investigations conducted as part of this project include four components which were carried out concurrently. The first component was the hydraulic studies to characterize the flow and dispersion characteristics of the river and of selected CSO's. The second was the intensive sampling of the river for three or more days following rainfall events. The third was continuous monitoring of DO and temperature. The final component was the sediment investigations which were initiated after the source of the DO impact was identified in previous studies (11).

HYDRAULIC STUDIES

As was discussed earlier in Section 3, the portion of the Milwaukee River in the study area is characterized by two hydraulically unique reaches. The upstream reach, stretching from Estabrook Park to the North Avenue Dam, is a relatively shallow, free flowing stream containing several areas with rapids. Conversely, the downstream reach from the North Avenue Dam to the river mouth is a deep, sluggish stream greatly influenced by the backwater effects of Lake Michigan. These distinct differences result in widely varying stream parameters, such as velocity, travel time, and dispersion. Variations in these parameters greatly influence the impact of both storm and CSO discharges in the Milwaukee River.

In order to investigate the stream characteristics, two types of dye studies were undertaken. The first type involved the monitoring of dye in both the upstream and downstream reaches of the river. This study enables the computation of actual instream dry weather velocities, travel times and dispersion coefficients and permitted quantitative comparison of the two reaches. The second set of dye studies was aimed at measuring CSO mixing characteristics in the river. Both studies have justified and verified several of the assumptions and coefficients used during the computer modeling. Likewise, they have illustrated several river flow and CSO mixing patterns and the sampling techniques required to adequately measure their interaction. In addition to the dye studies, velocity measurements near the sediments in the river were conducted to determine the scouring potential of river velocities and velocities due to CSO discharges.

Dye Sampling Program

Both the river and CSO dye studies were undertaken with Rhodamine WT dye. This dye was used since it is not affected appreciably by organics, and it is approved by the U.S. Geological Survey (14). For the river surveys, dye was injected as a single slug at Estabrook Park for the upstream reach and at the North Avenue Dam for the downstream reach. The CSO dye studies began by injecting dye in the outfall sewer at the intercepting devices (dividing structure for flow to treatment plant and overflow to river) to insure proper mixing before any backwater effects from the river were expressed. In both dye studies samples were taken at downstream bridges. Initial samples taken near the dye injection sites were gathered both cross-sectionally and with depth until complete lateral mixing was observed. The sampling at downstream sites was limited to surface and bottom samples at the center of the stream.

The actual sample gathering was done manually and with automatic sampling devices. Fluorometer readings taken at the laboratory were used to determine the dye concentrations. Sampling began early enough to capture the dye flow at each monitoring site and was continued until the entire dye plume had passed.

Instream Results

The first of three river dye studies began on November 8, 1976. Dye was injected at Estabrook Park and sampled at Capitol Drive and the North Avenue Dam. Figure 13 illustrates the monitored concentrations at the two sites. The last two studies were completed on the downstream reach of the Milwaukee River. On March 30, 1978, dye was injected at the North Avenue Dam with samples subsequently collected at Humboldt Avenue, Walnut Street, Kilbourn Boulevard, and St. Paul Avenue. On June 13, 1978, a second dye injection was made at the dam and samples were collected at Walnut Street, Wells Street and St. Paul Avenue. Figure 14 shows representative dye concentration curves from these injections.

Based on the sampling results, both the upstream and downstream reaches were divided into two smaller reaches for more detailed analysis. Using the methods outlined in Godfrey and Frederick (15) and Fisher (16) the mean velocities, travel times and dispersion coefficients of each reach were computed. The characteristics of the four reaches; Estabrook Park to Capitol Drive, Capitol Drive to North Avenue Dam, North Avenue Dam to Walnut Street and Walnut Street to St. Paul Avenue, for the three surveys are summarized in Table 2.

An examination of the overall trends shown in Table 2 supports the conditions that would be expected from a stream with the physical characteristics of the Milwaukee River. The reaches with the least downstream hindrance, Estabrook Park to Capitol Drive and North Avenue Dam to Walnut Street, have the highest velocities during both high and low flow conditions. The two reaches below these are influenced from backwater at North Avenue Dam and Lake Michigan and accordingly have lower velocities.

As would be expected, the backwater influence from Lake Michigan is less during high flows. During the June 13, 1978 survey, a flow of 215 cfs (6.1 m^3/sec) was recorded. The resulting velocity from Walnut Street to St. Paul Avenue was only 38 percent of the velocity from the North Avenue Dam to Walnut Street. This ratio increased to 51 percent during the 1700 cfs (48.1 m^3/sec) high flow condition of the March 30 survey.

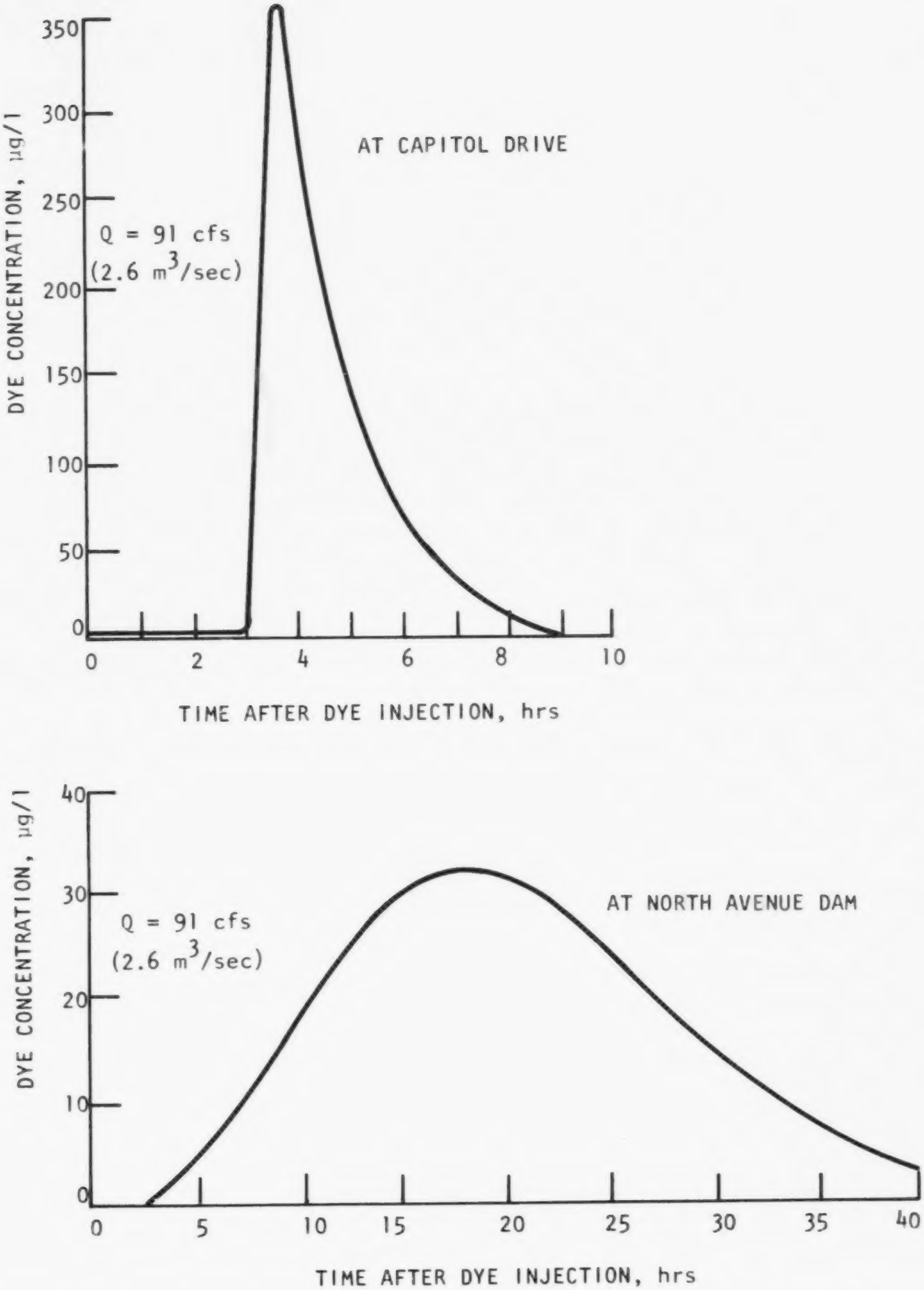


Figure 13. Monitored dye concentrations at Capitol Drive and North Avenue Dam during the November 8, 1976 Milwaukee River dye study.

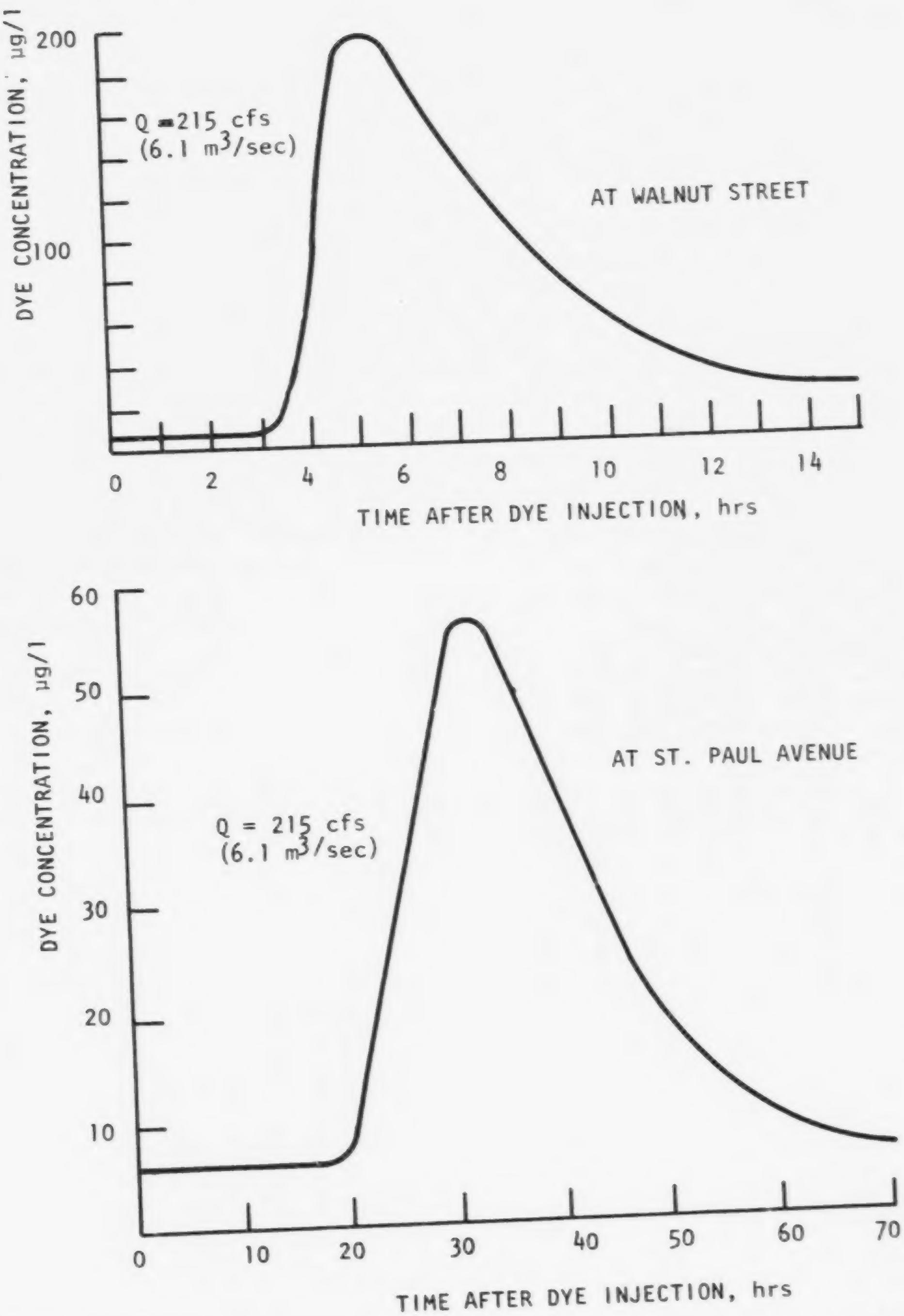


Figure 14. Monitored dye concentrations at Walnut Street and St. Paul Avenue during the June 13, 1978 Milwaukee River dye study.

TABLE 2. MILWAUKEE RIVER FLOW CHARACTERISTICS DETERMINED BY DYE TRACINGS

Tracing date	Flow[1] rate, cfs	Reach	Reach length, ft	Cross-sectional[2] area, ft^2	Estimated mean velocity, ft/sec	Monitored[3] mean velocity, ft/sec	Estimated travel time, hrs	Monitored[3] travel time, hrs	Longitudinal dispersion coefficients: Godfrey-Frederick, ft^2/sec	Longitudinal dispersion coefficients: Fischer, ft^2/sec
November 8, 1976	91	Estabrook Park-Capitol Drive	5,000	-	-	0.31	-	4.58	30	49
		Capitol Drive-North Ave. Dam	12,170	-	-	0.09	-	39.5	24	15
March 30, 1978	1700	North Ave. Dam-Walnut Street	4,650	2,380	0.71	0.78	1.82	1.67	30	78
		Walnut Street-St. Paul Ave.	6,750	3,230	0.53	0.40	3.54	4.67	50	124
June 13, 1978	215	North Ave. Dam-Walnut Street	4,650	2,380	0.09	0.16	14.4	8.08	46	63
		Walnut Street-St. Paul Ave.	6,750	3,230	0.07	0.06	26.8	29.2	4	1

[1] As measured at the USGS stream gauge at Estabrook Park.

[2] Average cross-sectional areas measured during typical flow conditions.

[3] Mean velocity and travel time based on the time between the centroids in the dye concentration curves measured at the sampling sites.

Conversions:
$cfs \times 0.02832 = m^3/sec$
$ft^2 \times 0.3048 = m$
$ft^2 \times 0.0929 = m^2$
$ft/sec \times 0.3048 = m/sec$
$ft^2/sec \times 0.0929 = m^2/sec$

The same trends exhibited in velocities are followed inversely by the computed tracer travel times; the greater the velocity a reach has the shorter its tracer travel time. As a rough check of the dye study velocities and travel times below the North Avenue Dam, estimated values were computed from the reach lengths and the continuity equation:

$$Q = VA$$

Where: Q = Flow rate.
V = Velocity.
A = Typical reach cross-sectional area.

The flow rates used in the continuity equation were measured upstream of the study area and were not adjusted for downstream flow contributions for two reasons. First, all of the river dye studies were conducted during dry weather and flow monitoring at the North Avenue Dam indicates that there is little discernible change in flow between Estabrook Park and the dam during these conditions. Secondly, there are no major tributaries to the river within the study area. The cross-sectional areas used for the estimated velocities and travel times were rough estimates and were not adjusted for the different flow rates. This is not a significant factor because the cross-sectional areas downstream of the North Avenue Dam are determined by the water level of Lake Michigan. Due to these factors, the estimated velocities and travel times can only serve as a yardstick with which to compare the dye study values. Nevertheless, the values compared well and indicate a reliable set of dye study data.

The most significant of the dye study results are the dispersion coefficients. These dispersion coefficients are a relative measure of the significance of longitudinal (or downstream) dispersion in the Milwaukee River. Longitudinal dispersion is defined as the action by which a flowing stream spreads out and dilutes a mass of pollutant (17). A pollutant such as CSO enters a stream and instead of traveling downstream as a slug, spreads itself out along the length of the stream as some parts travel faster and some slower than the mean flow velocity.

The rate of longitudinal dispersion is an important parameter in the modeling of pollutant loads to receiving streams. If longitudinal dispersion is significant, complex dispersion terms must be included in the modeling transport equations. The longitudinal dispersion coefficients for the Milwaukee River were computed over a wide range of flow conditions by the two well regarded methods presented earlier. The values are listed by individual reaches and flow conditions in Table 2.

The computed longitudinal dispersion coefficients are relatively low values. This indicates that longitudinal dispersion is a minor component of the total transport mechanism of the Milwaukee River and need not be modeled. A low longitudinal dispersion is typical of streams under the backwater, estuarial influence of a large body of water. The Clinch River in Tennessee is similar to the Milwaukee River in both width and depth, yet, as opposed to the Milwaukee River, it is a free flowing stream without backwater influences (15). In a dispersion study under similar discharges, the Clinch River had a longitudinal dispersion coefficient of 500 ft^2/sec (46.5 m^2/sec) at 1800 cfs (51.0 m^3/sec) and a coefficient of 150 ft^2/sec (13.9 m^2/sec) at 323 cfs (9.1 m^3/sec). Comparison with the values in Table 2 illustrates the minor influence of longitudinal dispersion in the Milwaukee River.

The relative importance of even a minor longitudinal dispersion transport does depend somewhat on the type of pollutant load to the river. An instantaneous loading comparable to an accidental fuel or toxic waste spill would illustrate significant longitudinal dispersion characteristics even in the Milwaukee River. However, in the study and modeling of CSO and storm sewer discharges which can last several hours and are distributed over a large stretch of the river, longitudinal dispersion would still be minor in comparison to convective transport.

CSO Results

The aim of the CSO dye studies was to measure the mixing capabilities of the CSO under different discharge magnitudes. Dye was added as a slug and the instream mixing was measured by sampling at a downstream bridge at three equidistant sites across the bridge and with depth. Dye was injected into the 96 in. (244 cm) outfall upstream of Walnut Street and into the 4x5 ft (1.2x1.5 m) outfall upstream of Cherry Street. Both outfalls are submerged and are located on the west bank of the river.

The two dye injections upstream of Walnut Street were made under low discharge velocities. Visual tracking of the dye showed no lateral mixing and the vast majority of the dye moved slowly down the west bank of the river. Subsequent analysis of the collected samples verified the visual conclusions. In fact, during the hour of sampling after each injection no dye at all was measured on the east side of the river. Only a minor concentration was found at mid-depth of the central sampling site during one injection. It is apparent that at low discharge flow rates mixing does not occur rapidly.

Rapid mixing was observed during the dye injection at the 4x5 ft (1.2x1.5 m) outfall. This outfall was discharging at a high rate and the dyed discharge plume could be seen traveling across the entire 250 ft (76.2 m) of channel width. This almost instantaneous lateral mixing was verified with the samples collected at Cherry Street.

The effect of the submerged outfalls is dependent upon discharge rates and the relative depth of the outfalls. Both outfalls studied are located slightly below the mid-depth of the channel. At the low discharge rates, most of the dye remains near the outfall and spreads fairly uniformly from mid-depth to the surface. Little discharge appears to affect the bottom. At the high discharge rates, the entire section from the bottom to the surface is influenced.

It follows that there is a general tendency for CSO to rise to the surface. Consequently, a surface outfall will tend to show less mixing and less bottom sediment resuspension than a submerged outfall in spite of similar high discharges. With most of the CSO outfalls below North Avenue Dam being submerged, good mixing of discharges, as well as river bottom sediment resuspension can be expected.

Velocity Measurements

Physical measurements of velocity in the river near the bottom sediments were taken in order to determine whether velocities due to increased river flow or CSO discharges are sufficient to scour these sediments. This was investigated due to the possible impact of resuspended sediments on DO levels in the river. The instrument utilized for these measurements recorded both velocity and direction so that the source of the changes in velocity could be determined.

Velocity measurements taken at a bridge located 300 feet (91 m) downstream from St. Paul Avenue indicate that the river velocities in the proximity of the sediments are generally quite low. During low flow conditions of about 100 cfs (2.8 m^3/sec) and following 0.30 in. (0.76 cm) of rainfall on May 31, 1977, the measured velocities were negligible. The velocities generally ranged from 0.0 to 0.25 ft/sec (0.0 to 0.08 m/sec), however, the direction of this velocity oscillated between the upstream and downstream directions. Following the 0.71 in. (1.80 cm) of rainfall on August 3 through August 5, 1977 and river flows reaching 498 cfs (14.1 m^3/sec) at the North Avenue Dam, there was still very little measurable velocity near the bottom sediments in the lower river. Values during this survey occasionally were as high as 0.40 ft/sec (1.2 m/sec) but were generally in the range of 0.0 to 0.25 ft/sec (0.0 to 0.8 m/sec). The direction of these velocities were quite variable as during the previous survey. This is understandable since the velocity near the bottom sediments is generally estimated as 25 percent of the mean stream velocity (18).

Velocity measurements in the vicinity of submerged CSO outfalls showed different results. Two outfall locations were monitored approximately 15 ft (4.6 m) from the outfall in the river and about 1.0 ft (0.30 m) above the sediment surface. An outfall located in the east bank of the Milwaukee River at St. Paul Avenue was monitored for ten separate events. This outfall is a completely submerged 4.0x8.5 ft (1.2x2.6 m) box. A second outfall located just upstream of Walnut Street on the west bank of the river was monitored for six storm events. This outfall is also completely submerged having a diameter of 8.0 ft (2.4 m). The invert of the pipe entering the river is approximately 12 ft (2.7 m) below the water surface.

Following an extremely intense rainfall event on August 3, 1977, a velocity of about 12.0 ft/sec (3.7 m/sec) was measured near the outfall at St. Paul Avenue. There was 0.22 in. (0.56 cm) of rainfall during a 10 minute period in the vicinity of this site. On July 31, 1978 a velocity of over 5.0 ft/sec (1.5 m/sec) was measured at the Walnut Street outfall. Approximately 0.40 in. (1.01 cm) of rainfall was recorded during a 10 minute time period during this storm. Both of these measurements were made approximately 1.0 ft (0.3 m) above the sediment surface. There was severe clogging of the velocity and direction meter during both of these events which may have resulted in lower measured values than the actual velocities. The force of the discharges was so large as to actually push the velocity and direction meter out away from the outfall and lift it off the bottom. Other measurements of velocity generally ranged from 1.0 to 5.0 ft/sec (0.3 to 1.5 m/sec). This data indicates that there is considerable potential from submerged CSO outfalls to scour river sediments.

INTENSIVE SURVEYS

Each of the intensive sampling surveys is related to the volume, duration and distribution of rainfall in the study area. Other factors that were evaluated include the locations in the river where the impacts are present, the river flow as related to the magnitude and duration of the impacts, and the prestorm conditions which affect the magnitude of the impacts. Because of the ice covered conditions of the Milwaukee River during the winter, only the warm weather periods were evaluated in the monitoring program.

The six intensive surveys conducted during this project were used to define the changes in the quality of the Milwaukee River at five locations shown in Figure 2. These locations are described as follows:

1. Capitol Drive - Port Washington Road: This site is situated immediately upstream of the CSO area and is used to quantify the incoming loads to the study area. Survey I uses the Capitol Drive location while subsequent surveys use Port Washington Road because of a more accessible location.

2. North Avenue: The second monitoring location is situated immediately above the North Avenue Dam, providing data on those portions of the river that are influenced by CSO but are above the lake inflows. A continuous DO and temperature monitor is located here.

3. Walnut Street: This monitoring site is situated within the lake influenced areas of the river approximately 4600 ft (1400 m) downstream of the dam.

4. Wells Street: Another site within the lower portions of the river.

5. St. Paul Avenue: This site is the final location within the lake influenced portions of the river which lies immediately above the confluence of the Menomonee River. A continuous DO and temperature monitor is located at this site.

The intensive surveys were conducted for a period of three to five days with samples taken at intervals from three to six hours to try and quantify the changes in water quality after various rainfall events. One survey was conducted during dry weather to establish the base line conditions which exist in the study area after the effects of previous wet weather discharges have abated. All samples for chemical analysis were collected from bridges in cross section and composited before being transported to the laboratory for analysis. Fecal coliform samples were collected at mid-channel approximately 2 ft (0.6 m) below the water surface. Flow measurements used in the surveys were available from the USGS gauge located at Estabrook Park and the stage recorder installed at the North Avenue Dam during the final year of the project. Each of the surveys will be individually described in the remaining portions of this section. Analysis of the data from these surveys is discussed in Section VI. Data summaries for these surveys are listed in Tables A-1 through A-13 in the Appendix.

Survey I - Dry Weather Flow

The first intensive monitoring survey was conducted on September 21-23, 1976 to define the dry weather conditions within the CSO influenced portions of the river. Table 3 lists the rainfall and river flow conditions during each of the six intensive surveys that are part of the project. Survey I was an ideal dry weather survey because the flows in the Milwaukee River were extremely low after a long, dry summer season. Figure 15 presents the DO conditions monitored at the five locations previously described for the intensive surveys. The DO levels observed at the upstream boundary (Capitol Drive) and at North Avenue range from 5.7 to 9.1 mg/l. These values represent 60 and 95 percent satuation, respectively. The physical characteristics of the upper reach of the river tend to make the trends in DO at these two river sites appear quite similer. As shown previously in the cross section figures of Section 3, the upper portions of the river are extremely shallow. This results in substantial growth of macrophytic plants which cause the observed diurnal variations in DO at these sites.

In the lower portions of the river, the DO levels are significantly depressed when compared to the upper reaches. At Walnut Street, which is approximately 4600 ft (1400 m) downstream of North Avenue, the DO values ranged from 1.0 to 4.0 mg/l less than the values measured in the upper river. At St. Paul Avenue, the DO levels were less than 2.0 mg/l during most of the survey. Also, there is only a slight diurnal variation in the DO values in the lower river during this survey. This portion of the river is much deeper than upstream and thus, any changes in DO due to photosynehetic activity must be due to phytoplankton. Measured secchi depths range from approximately 18 to 30 in. (45.7 to 76.2 cm) and measured chlorophyll a values indicate that a substantial population of phytoplankton are present at the time of the year this survey was conducted (Appendix Table 14). The short photoperiod in September and the depth of the river precludes any observable impact on DO levels.

The fecal coliform data of Figure 16 indicates that residual effects of previous overflows from upstream were present during this survey. The trend of decreasing coliform counts at four monitoring locations as the survey progressed indicates that these sites could have been influenced by upstream discharges that occurred during a rainfall that did not cover the study area. The Walnut Street data does not show this relationship,

TABLE 3. SUMMARY OF RAINFALL AND RIVER FLOW CONDITIONS FOR INTENSIVE MONITORING SURVEYS

			Average rainfall			
Survey number	Date survey initiation	Survey duration, days	Total survey volume, in. (cm)	Duration, hrs	Antecedent dry period, days	Peak flow at Estabrook Park, cfs (m3/sec)
1	9-21-76	3	0.0	0.0	11	85 (2.41)
2	5-31-77	3	0.30 (0.76)	1.70	40	154 (4.36)
3	6-18-77	3	0.54 (1.37)	0.97	6	955 (27.05)
4	8-3-77	5	0.71 (1.80)	3.19	10	498 (14.10)
5	6-16-78	4	1.86 (4.72)	8.80	4	3112 (88.13)
6	7-26-78	3	0.63 (1.60)	1.84	5	599 (16.96)

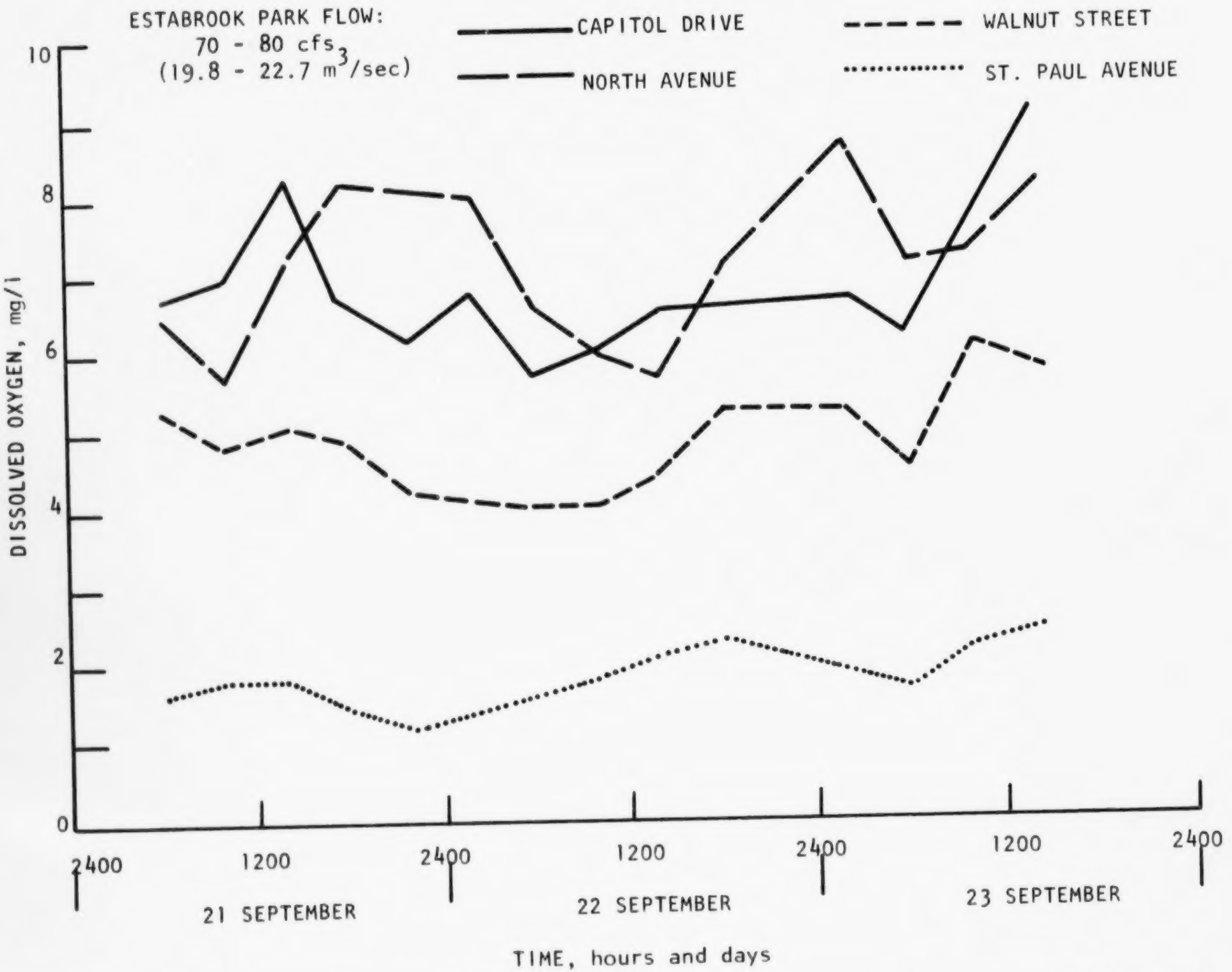


Figure 15. Monitored DO levels in the Milwaukee River during dry weather, September 21-23, 1976 (Survey 1).

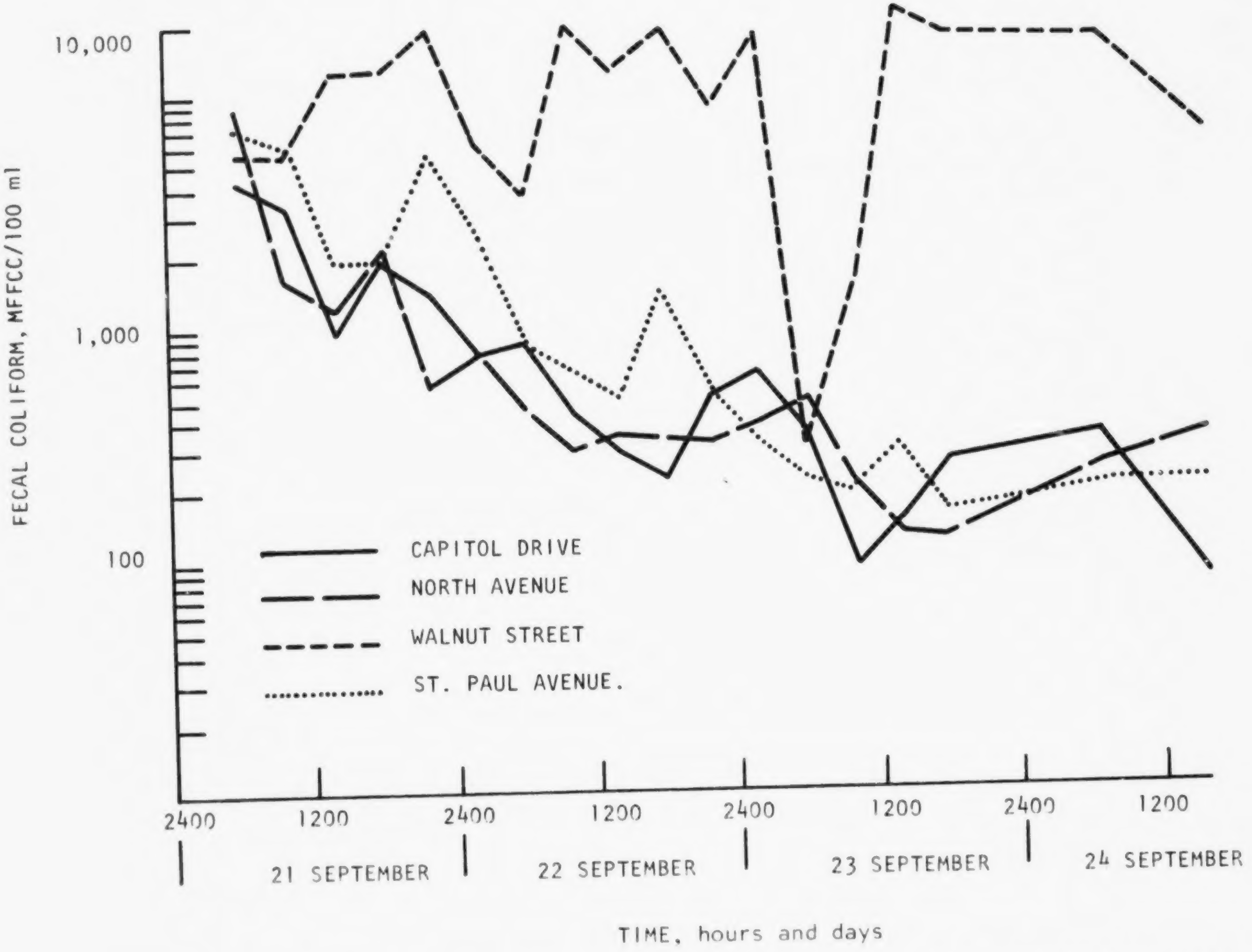


Figure 16. Monitored fecal coliform levels in the Milwaukee River during dry weather, September 21-24, 1976 (Survey 1).

but rather shows a continuous high level of coliforms except for one value on September 23rd. No sanitary overflows are known to exist in this area but an obvious source of fecal coliforms must be present to provide these high values. Dry weather surveys conducted previously as part of the Milwaukee CSO project (11) did not show these same elevated values. The final day of the survey shows the coliform concentrations leveling off to approximately 200 counts per 100 ml. The high counts at the upstream boundary of the CSO area throughout the survey seems to reinforce the concept that a storm passed through the upper portions of the basin without affecting the study area raingauges.

Survey 2 - Wet Weather Flow

The first wet weather data collected as part of this project was during Survey 2 which took place on May 31 to June 2, 1977. The survey was initiated as 0.30 in. (0.76 cm) of rainfall was recorded for the area on May 31. The river flows before and during the survey were again extremely low for this event because of the long dry period before this survey. The previous rainfall event which was greater than 0.1 in. (0.25 cm) was recorded on April 20, 1977 which means approximately 40 days of dry weather preceeded this event. The DO and fecal coliform conditions are presented in Figures 17 and 18. The upstream boundary station for the wet weather surveys was changed to Port Washington Road rather than Capitol Drive because of accessibility and the location of a combined sewer at the Capitol Drive location which could affect the sampling data. The DO data illustrated for the North Avenue Dam and St. Paul Avenue sites prior to the storm event on May 31 was obtained from the continuous DO and temperature monitors.

The DO data of Figure 17 shows the upstream DO to range from 5.2 to 11.0 mg/l. This represents approximately 60 and 120 percent of saturation with a major portion of the variability being associated with photosynthesis. On June 1, there is little variation in DO with the values being around 6.0 mg/l which is a result of the 100 percent cloud cover and two percent sunshine on this day. At the North Avenue Dam site, there was a rapid decline in DO during the first 8 hours after the rainfall event followed by a steady recovery in DO for the remainder of the survey. The DO at this location was below the 5.0 mg/l standard for portions of all three days of this survey. The pre-storm DO data was obtained from the continuous DO monitor.

At Walnut Street, the DO values were in the range of 2.3 to 3.9 mg/l during the entire survey. Wet weather impacts are difficult to discern at this site since the DO values are only slightly lower than observed in the dry weather survey. The Wells Street location exhibited a decline of DO from 2.6 to nearly 0.0 mg/l within 18 hours after the rainfall. The DO values remained below 1.0 mg/l until late on June 2. Recovery of DO after this event was extremely slow due to the low river flow conditions. The St. Paul Avenue site exhibited DO levels that were between 1.0 and 3.7 mg/l. For much of the survey the DO at this site was in the same range as at Walnut Street. This observation was probably due to the effect of Lake Michigan in the lower reaches of the river. It must be mentioned that the DO values of Figure 17 were averaged over depth for each location. At Walnut

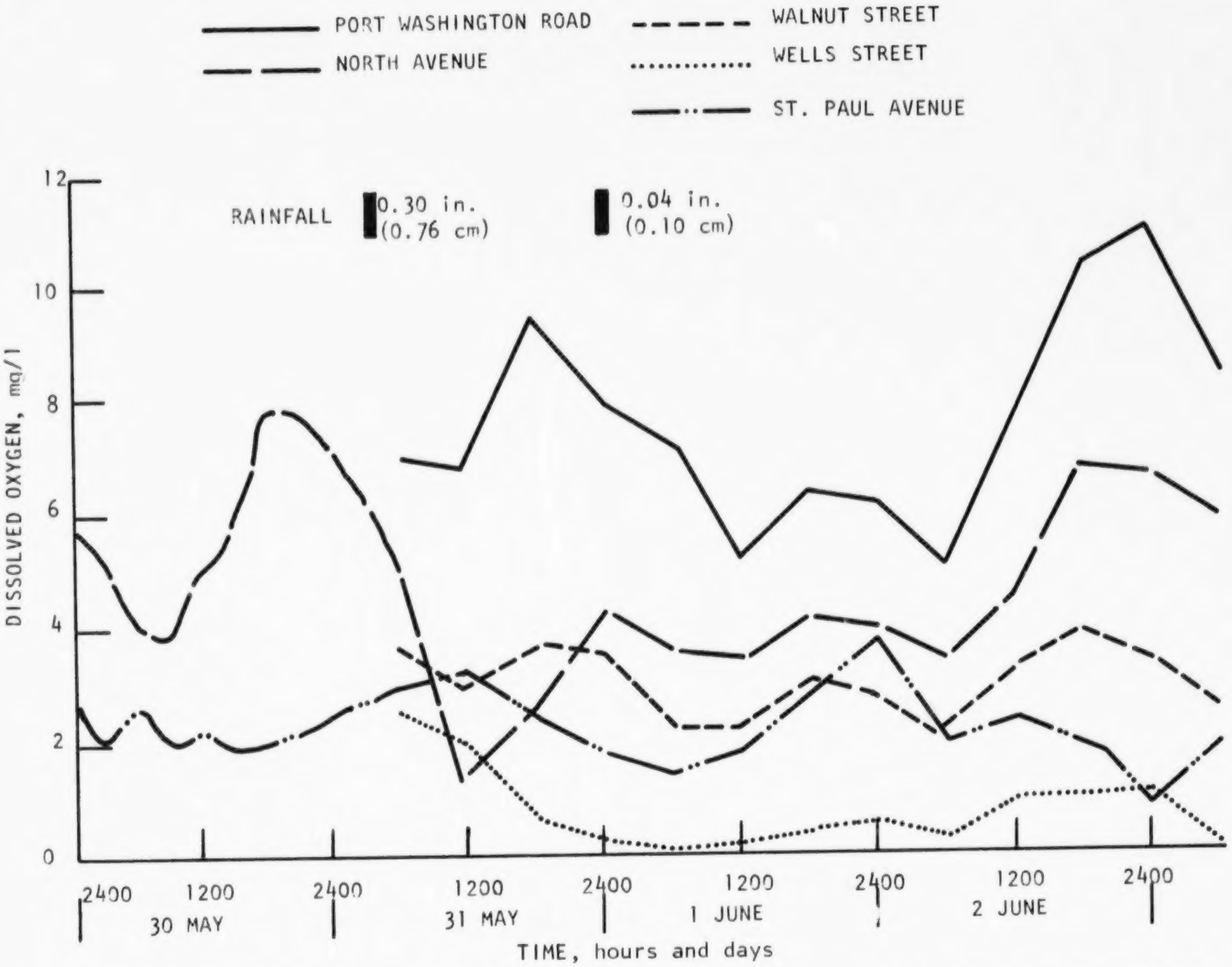


Figure 17. Monitored DO levels in the Milwaukee River during wet weather, May 31-June 2, 1977 (Survey 2).

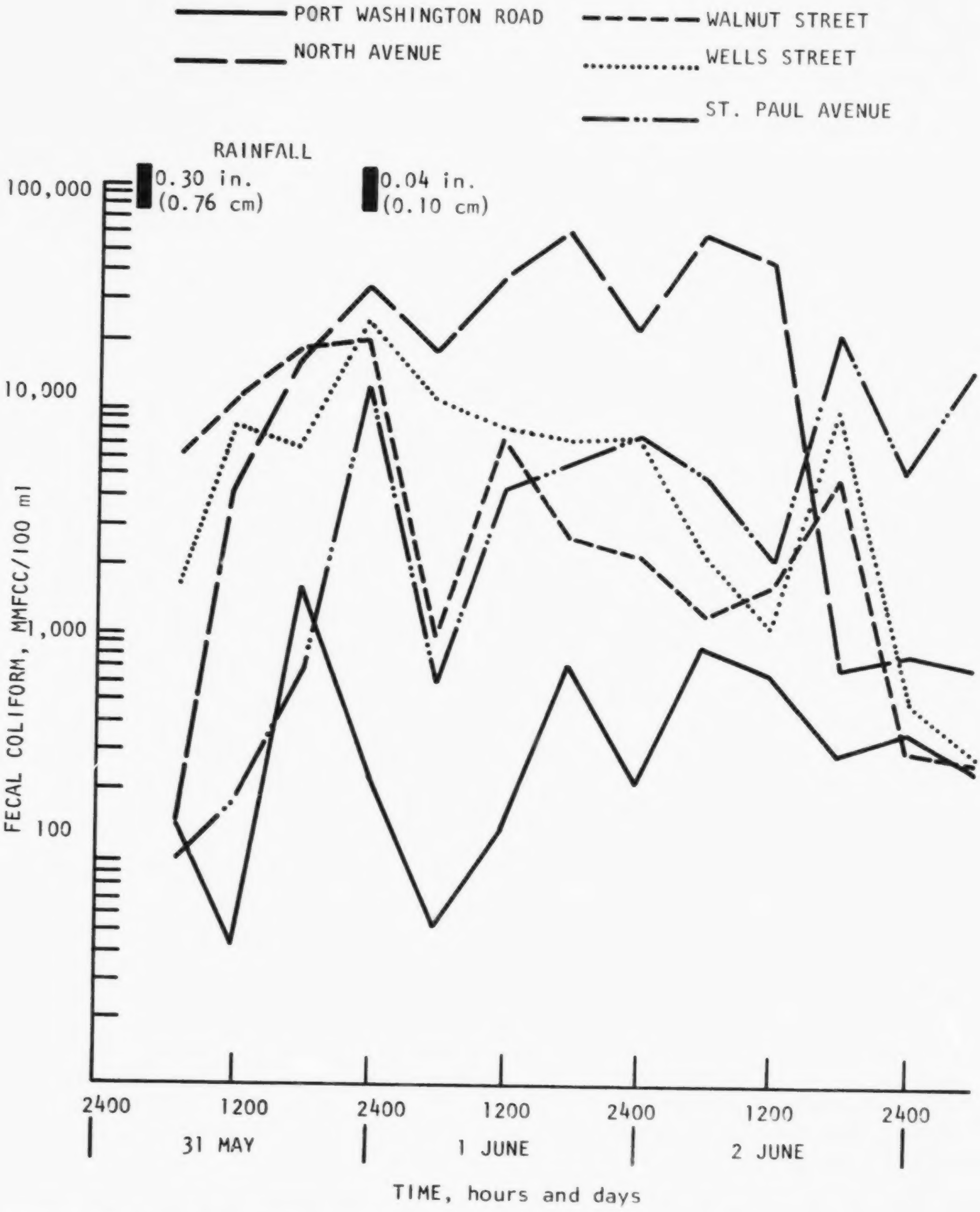


Figure 18. Monitored fecal coliform levels in the Milwaukee River during wet weather, May 31-June 2, 1977 (Survey 2).

Street the DO only varied by less than 1.0 mg/l with depth while at St. Paul Avenue the variation was as high as 5.0 mg/l. Significant temperature variations were also found at this location which indicates the cooler, high DO lake water is probably masking the normal DO sag for this location.

The fecal coliform results for Survey 2 are presented in Figure 18. The most noticeable feature of this figure is the very low counts at the upstream boundary of the CSO area throughout the duration of the survey. The North Avenue Dam site exhibits the highest concentrations during the first two days of the survey but then rapidly decline on the third day to levels very close to the upstream boundary (Port Washington Road). St. Paul Avenue remains fairly low in fecal coliform levels throughout the first two days of the survey and then rapidly increase to much higher levels. The low flow conditions during this event could have allowed the lake influence to be much more pronounced at this location, thus diluting the coliform levels. The remaining downstream sites are fairly consistent throughout the survey with values in the 10^2 to 10^3 range which are an order of magnitude less than at the North Avenue site. Again, the lake influence on the Walnut and Wells sites would tend to dilute the coliform concentrations even though a majority of the CSO discharges occur below the North Avenue Dam.

Survey 3 - Wet Weather Flow

The second wet weather event was conducted on June 18 through June 21, 1977. This survey is an example of a wet weather event which included a substantial rainfall but little observable impact on DO levels in the river. The rainfall data from this survey is unique in that the average volume of rain which fell in the Milwaukee area was 0.54 in. (1.37 cm) with one gauge recording 0.50 in. (1.27 cm) in a 15 minute period. The average daily river flows during this survey ranged from 154 to 256 cfs (4.4 to 7.2 m^3/sec) with the peak river flow occurring approximately 3 hours after the rainfall event ended with a flow of 955 cfs (27 m^3/sec) at Estabrook Park.

The DO conditions in the river during this survey are presented in Figure 19. These indicate the same trends of decreasing DO concentrations with distance downstream as observed during Surveys 1 and 2. At Port Washington Road the DO levels were between 8.0 and 13.0 mg/l throughout the survey. The North Avenue site exhibited a large diurnal variation in DO with the values ranging from 54 to 160 percent of saturation. The sites in the lower river also exhibited the same variation in DO, however, this variation becomes more attenuated with distance downstream.

The data from the continuous DO monitors at North Avenue and St. Paul provide the change in conditions before the rainfall event began. The North Avenue site shows only a slight decrease in the diurnal peaks due to the storm event. St. Paul Avenue presents a general decrease in DO from prestorm to poststorm conditions.

The fecal coliform results for this survey are presented in Figure 20. At all monitoring sites, the coliform concentrations rapidly decrease during the second and third day of the survey. The upstream locations at Port Washington

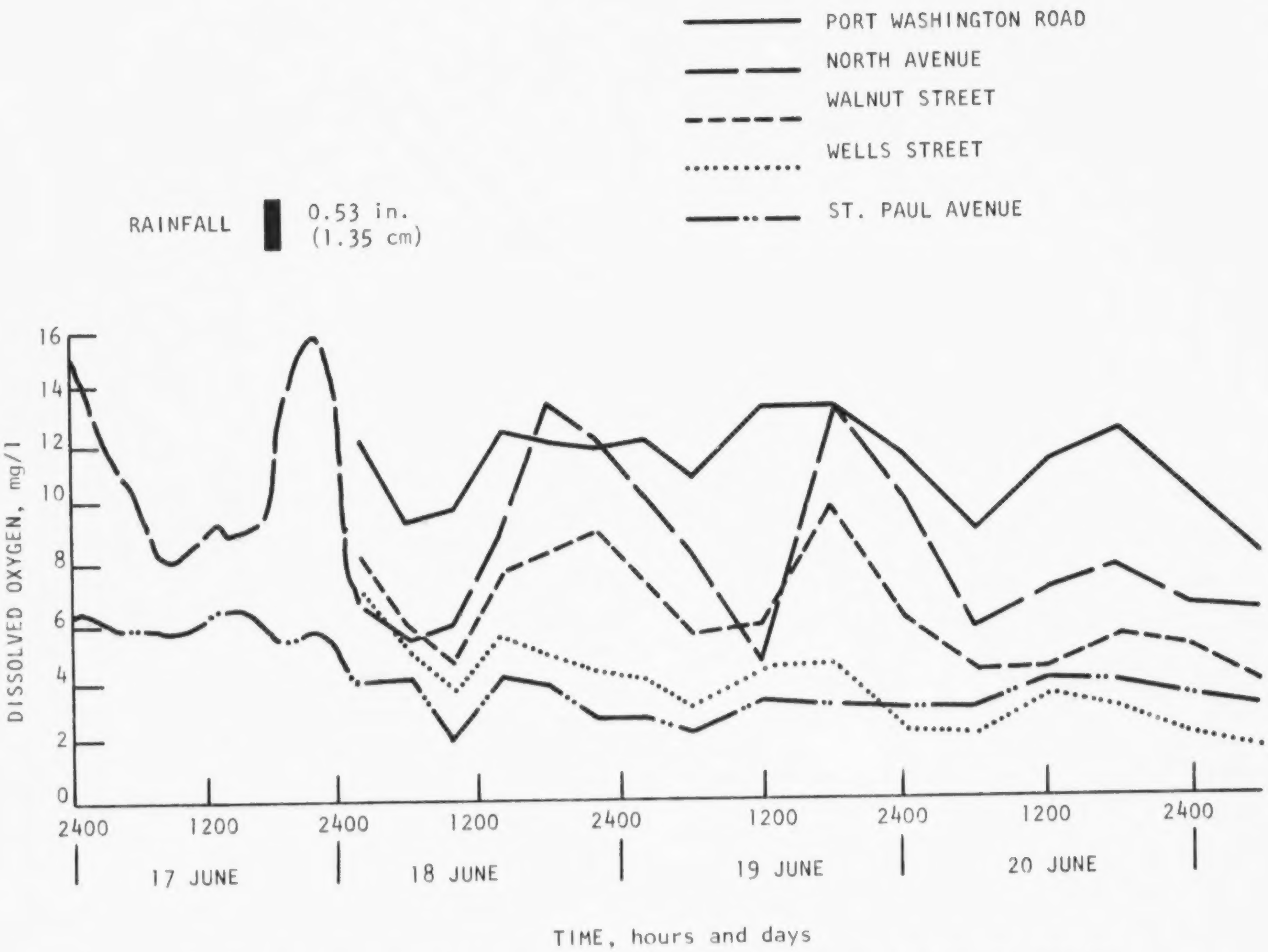


Figure 19. Monitored DO levels in the Milwaukee River during wet weather, June 17-20, 1977 (Survey 3).

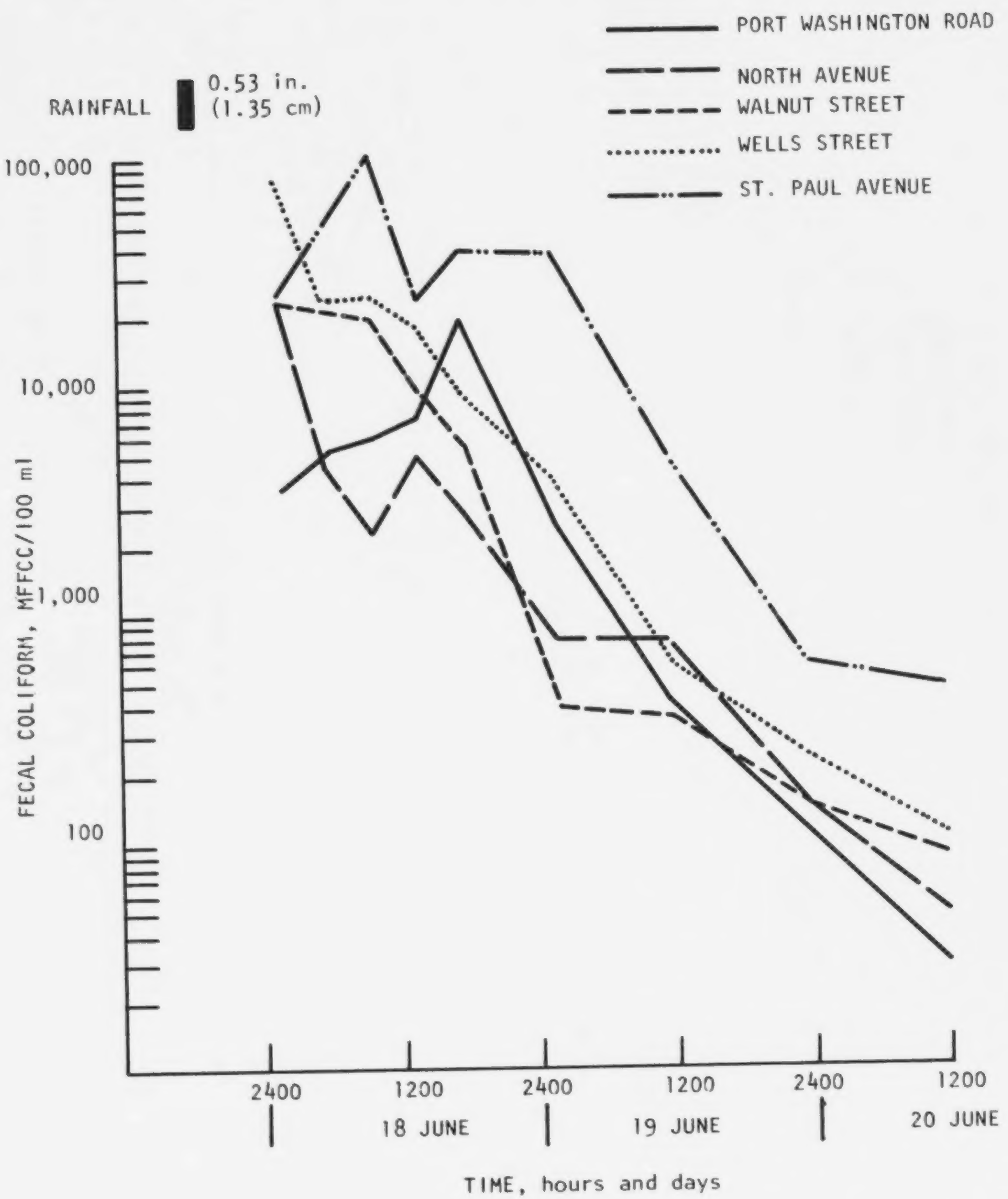


Figure 20. Monitored fecal coliform levels in the Milwaukee River during wet weather, June 18-20, 1977 (Survey 3).

Road and the North Avenue Dam generally have the lowest concentrations during the survey. The Port Washington Road location shows a rapid rise in count during the first day of the survey which might be a function of cross-connected storm sewers in the upper portions of the Milwaukee River drainage area. Throughout the duration of this survey, the St. Paul Avenue site contains the highest concentrations of fecal coliforms. This is the exact opposite of Survey 2 when it exhibited the lowest levels. The difference may be due to the higher river flows present during Survey 3. With these conditions, the lake influence was not as prominent as in the past events. One other point of interest in this survey is the very low values for the Walnut Street site during the second day of the survey.

Survey 4 - Wet Weather Flow

This survey is unique to the remaining wet weather monitoring events because of the continuous rainfall that was recorded on different days throughout the five days of the survey. The initial rainfall on the study area on August 3, 1977 averaged about 0.33 in. (0.84 cm). Additional rainfall volumes are listed in Table 4 for the remaining days of the survey. The DO data from this survey is presented in Figure 21 and in a slightly different manner in Figure 22 in order to show the longitudinal trends. The North Avenue Dam site had fluctuating DO levels between 2.0 to 5.0 mg/l during the first day (August 4) after the start of the survey. The DO recovered on August 5 and did not violate the 5.0 mg/l standard for the remainder of the survey. At the lower river sites, the DO decreased progressively during the first 12 hours of the survey and then dropped to nearly 1.0 mg/l at these lower river sites by 1800 hours on August 4th. This drop to 1.0 mg/l is likely due to a second rainfall and overflow event which occurred at 1500 hours on August 4th.

Between the Walnut Street and Cherry Street sites which is a distance of approximately 1125 ft (343 m), the observed differences in DO levels was as much as 5.5 mg/l at 1800 hours on August 4th. Downstream from the Cherry Street location, the DO remained near 0.0 mg/l for nearly a full day. Two additional storms on August 5th further suppressed the DO in the lower river until August 6th when the DO recovered quite dramatically. Much of the recovery can be attributed to the change in flow rate in the river. Table 5 lists the average daily flow and range in flow at the North Avenue Dam and Estabrook Park during this survey. The high flows towards the end of the survey may account for the rapid DO recovery observed in the lower river sites.

The fecal coliform data for this survey is presented in Figure 23. Similar trends as the previous wet weather survey are exhibited in this figure with the St. Paul Avenue site showing the highest coliform levels and the upstream boundary of the CSO area having the lowest levels. The rainfall throughout the survey maintains the levels of fecal coliforms and prevents the flushing of the coliforms by the higher than normal river flows which are present during the final portions of the survey.

TABLE 4. RAINFALL DISTRIBUTION IN CSO AREA DURING THE WET WEATHER INTENSIVE SURVEY ON AUGUST 3-8, 1977 (SURVEY 4)[1]

		Broadway Street		Holton Street		Bartlett Avenue	
Date, 1977	Time	Rainfall volume, in. (cm)	Duration, hours	Rainfall volume, in. (cm)	Duration, hours	Rainfall volume, in. (cm)	Duration, hours
8/3	2130	0.36 (0.91)	0.75	0.50 (1.27)	0.83	0.14 (0.36)	0.83
8/4	1515	0.23 (0.58)	0.75	0.20 (0.51)	0.83	0.08 (0.20)	1.17
8/5	1145	0.34 (0.86)	0.25	0.03 (0.08)	0.50	0.00	-
8/5	1420	0.20 (0.51)	1.18	NA[2]	-	0.00	-

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[1] Location of raingauges is illustrated in Figure 2.

[2] Not available.

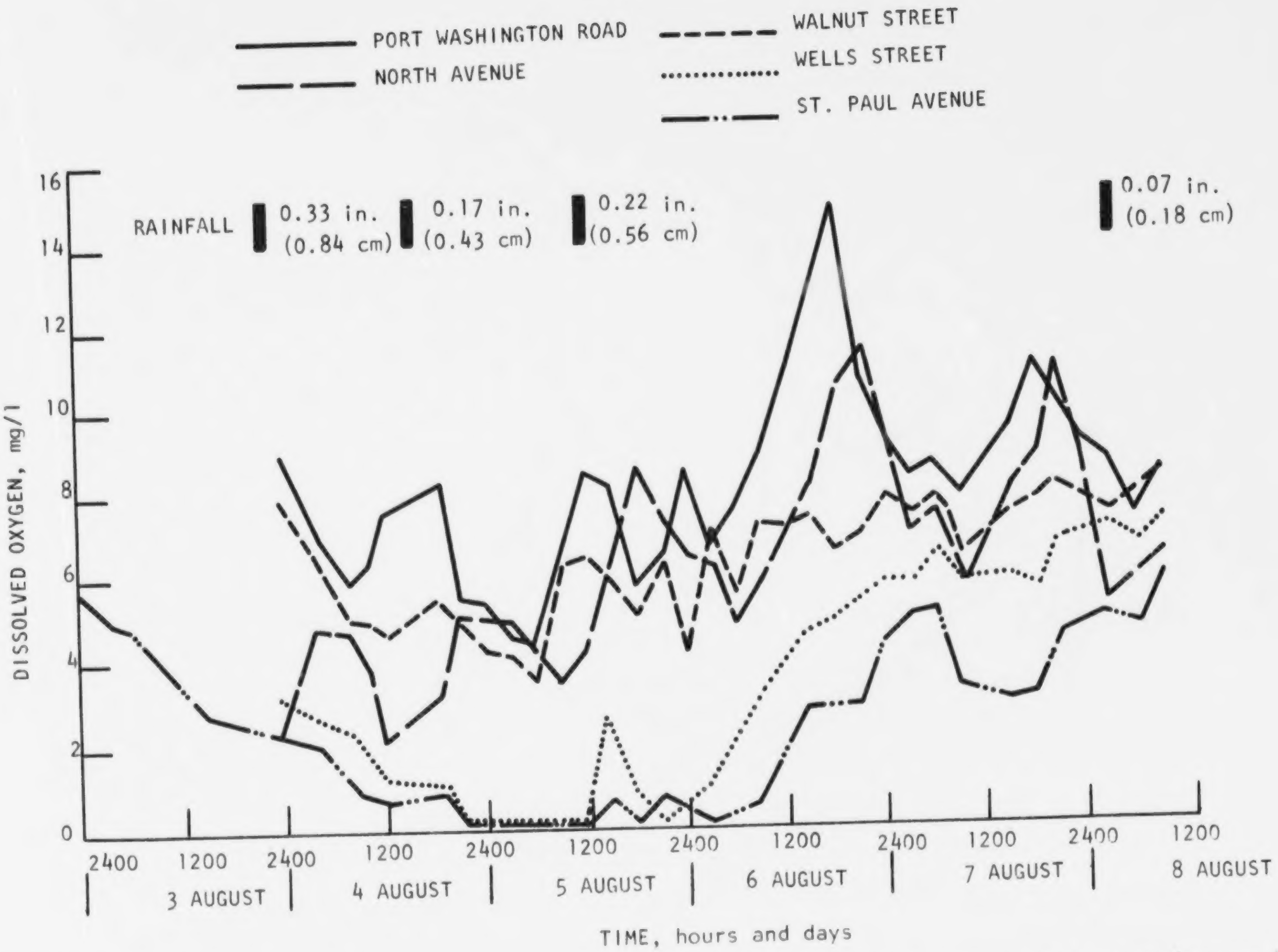


Figure 21. Monitored DO levels in the Milwaukee River during wet weather, August 3-8, 1977 (Survey 4).

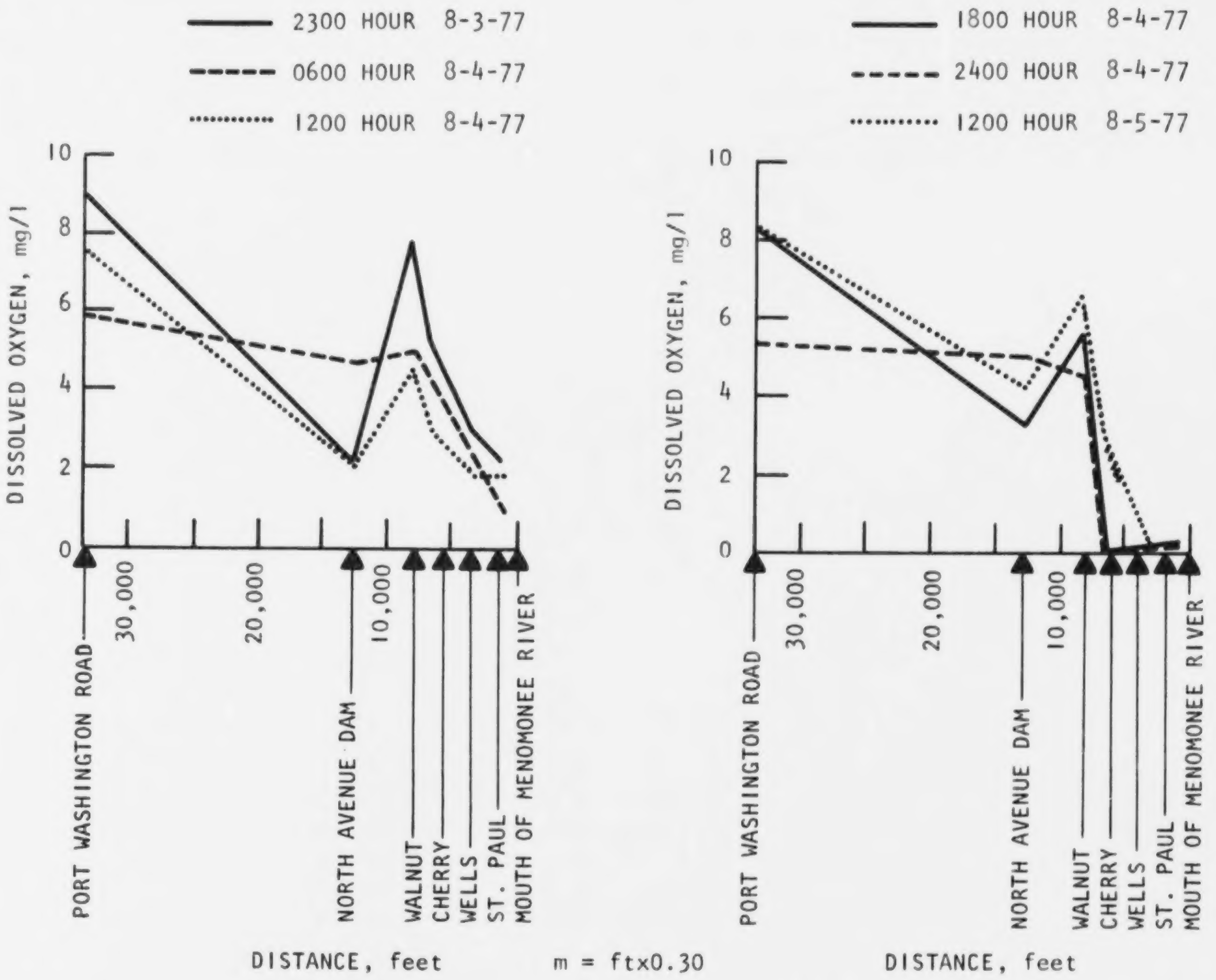
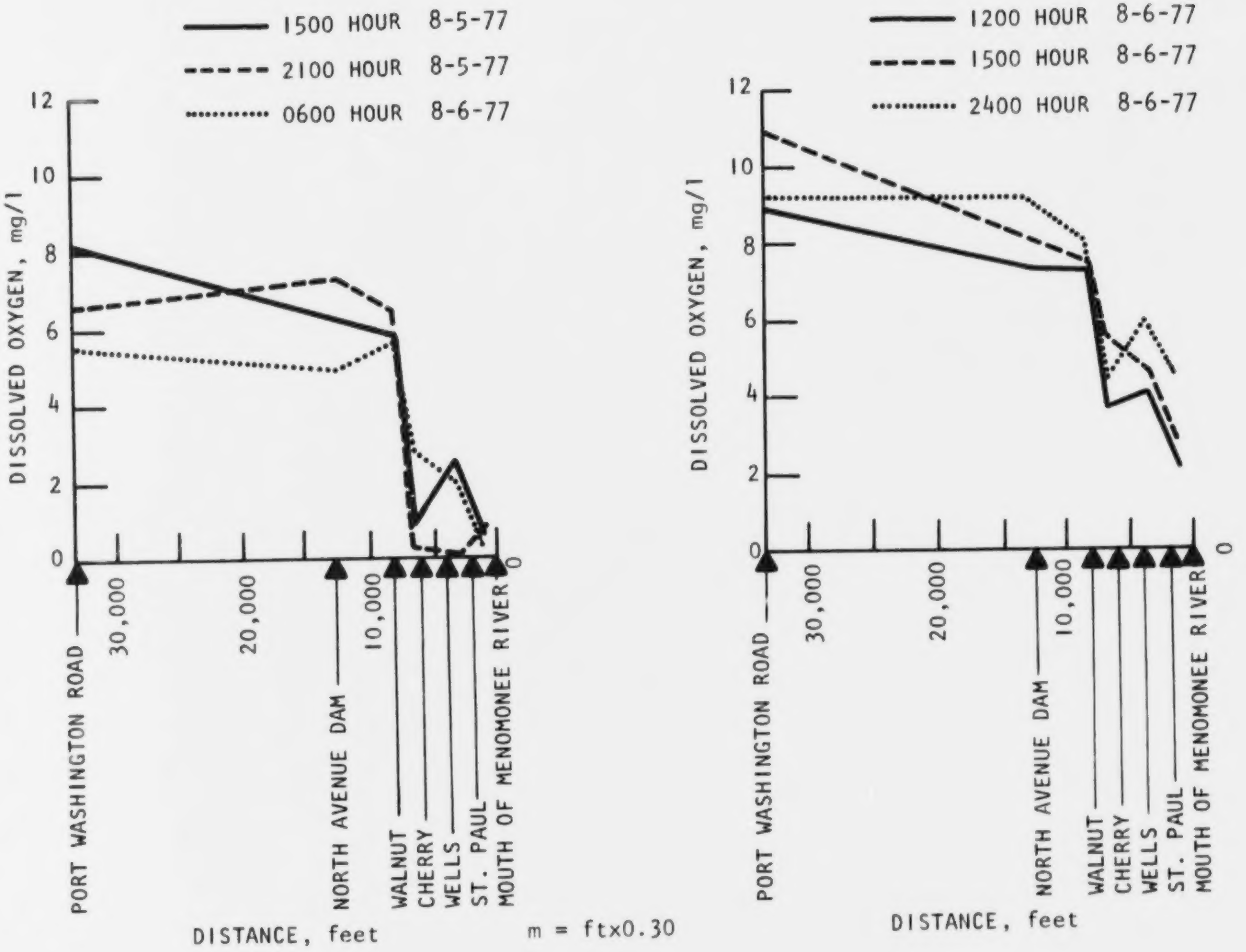


Figure 22. Monitored DO versus distance in the Milwaukee River during wet weather, August 3-6, 1977 (Survey 4) (continued).



m = ftx0.30

Figure 22 (continued).

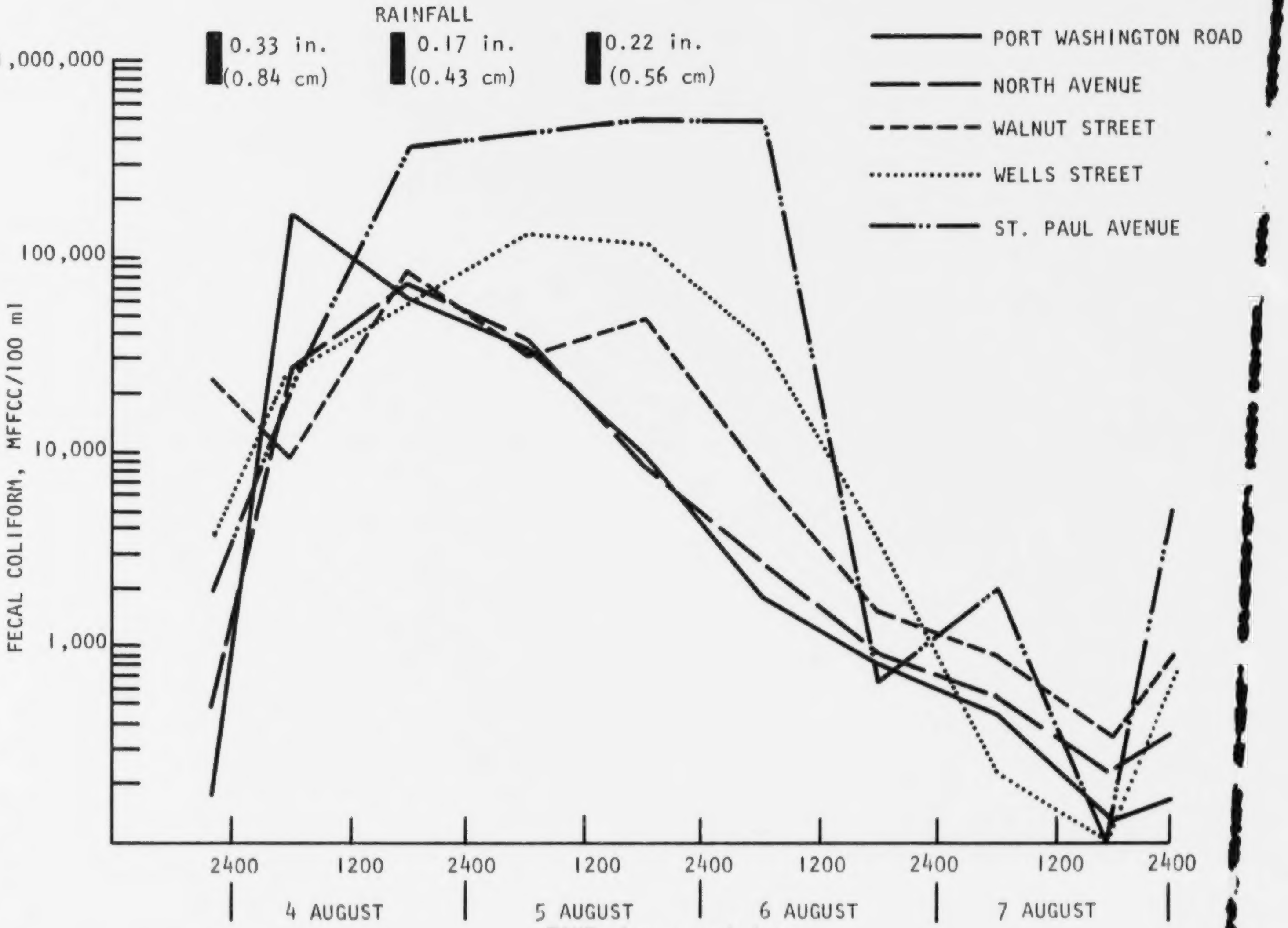


Figure 23. Monitored fecal coliform levels in the Milwaukee River during wet weather, August 3-7, 1978 (Survey 4).

TABLE 5. VARIATIONS IN RIVER FLOW AT THE NORTH AVENUE DAM AND ESTABROOK PARK DURING THE INTENSIVE MONITORING SURVEY ON AUGUST 3-8, 1977 (SURVEY 4)

	North Avenue Dam		Estabrook Park	
Date 1977	Average flow, cfs (m^3/sec)	Range in flow, cfs (m^3/sec)	Average flow, cfs (m^3/sec)	Range in flow, cfs (m^3/sec)
8/3	145 (4.1)	97-523 (2.7-14.8)	130 (3.7)	105-611 (3.0-17.3)
8/4	220 (6.2)	112-534 (3.2-15.1)	170 (4.8)	98-418 (2.8-11.8)
8/5	371 (10.5)	178-440 (5.0-12.5)	344 (9.7)	176-405 (5.0-11.5)
8/6	479 (13.6)	342-522 (9.7-14.8)	461 (13.1)	312-498 (8.8-14.1)
8/7	487 (13.8	441-514 (12.5-14.6)	448 (12.7)	399-491 (11.3-13.9)
8/8	419 (11.9)	371-507 (10.5-14.4)	383 (10.8)	334-498 (9.5-14.1)

Survey 5 - Wet Weather Flow

In order to quantify the effects of wet weather discharges on the Milwaukee River, the fifth survey was undertaken on June 16, 1978 while the flow in the river reached over 3000 cfs (84 m^3/sec). The average river flows during the survey were 453 cfs (12.7 m^3/sec) on June 16th, 1623 cfs (45.4 m^3/sec) on June 17th and 1645 cfs (46 m^3/sec) on June 18th. The DO values in the river during this survey are presented in Figure 24. An average of 1.86 in. (4.72 cm) of rain fell on the study area during the survey with one gauge registering 2.3 in. (5.8 cm). The resulting DO values in the river show very little sag from the start to the end of the survey. By the third day of the survey each monitoring location had approximately the same DO level. These results are likely due to the rapid rate at which the water was transported through the study area during the high flow conditions.

The fecal coliform concentrations in the river during Survey 5 are presented in Figure 25. The high flows that occur during this survey seem to have less of an impact on the coliform levels than in previous surveys. Again the upstream sites show the lowest concentrations but they remain relatively constant after the first day of the survey. The highest concentrations were obtained at the Wells Street site especially during day 3 of the

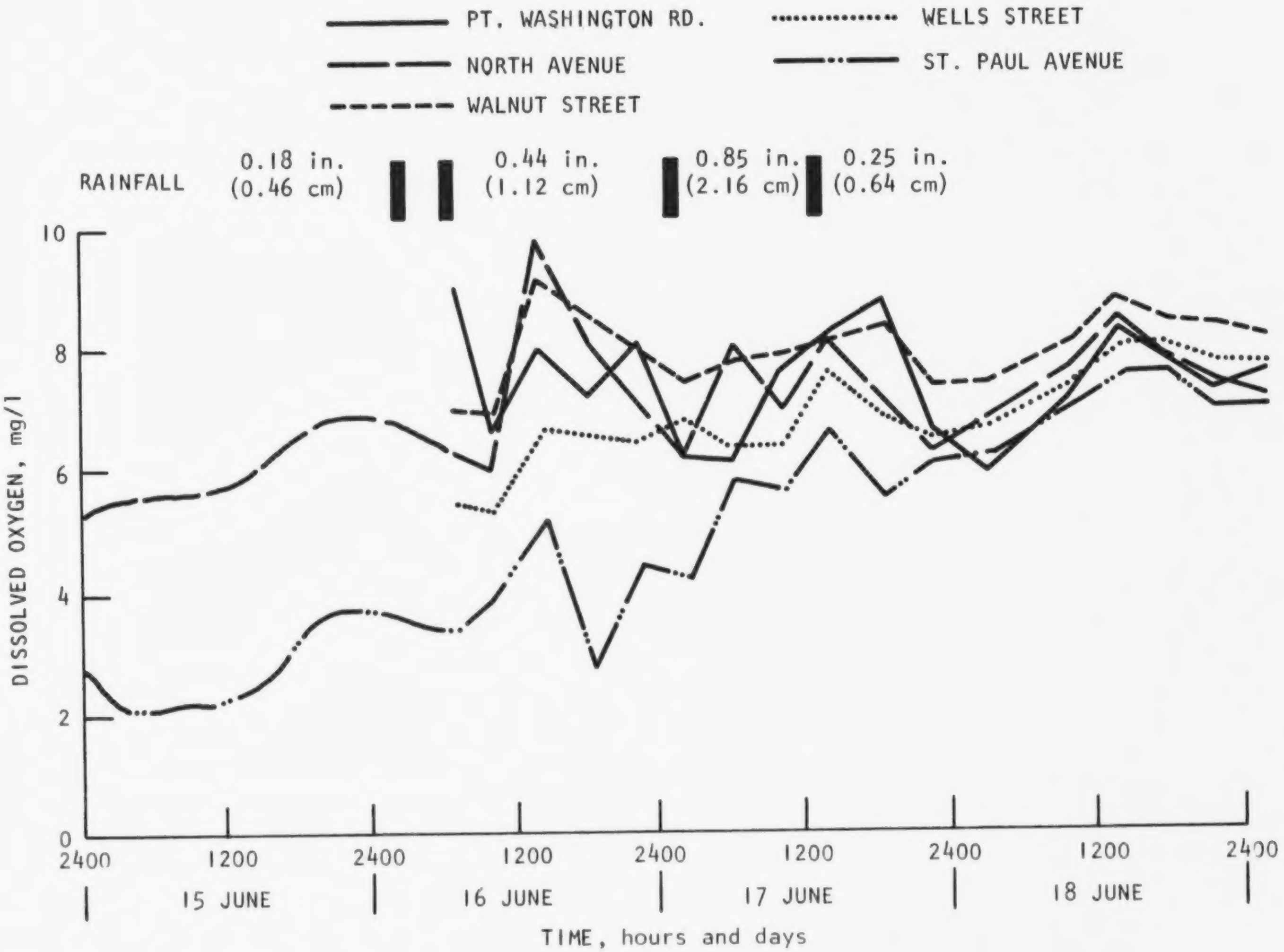


Figure 24. Monitored DO levels in the Milwaukee River during wet weather, June 15-18, 1978 (Survey 5).

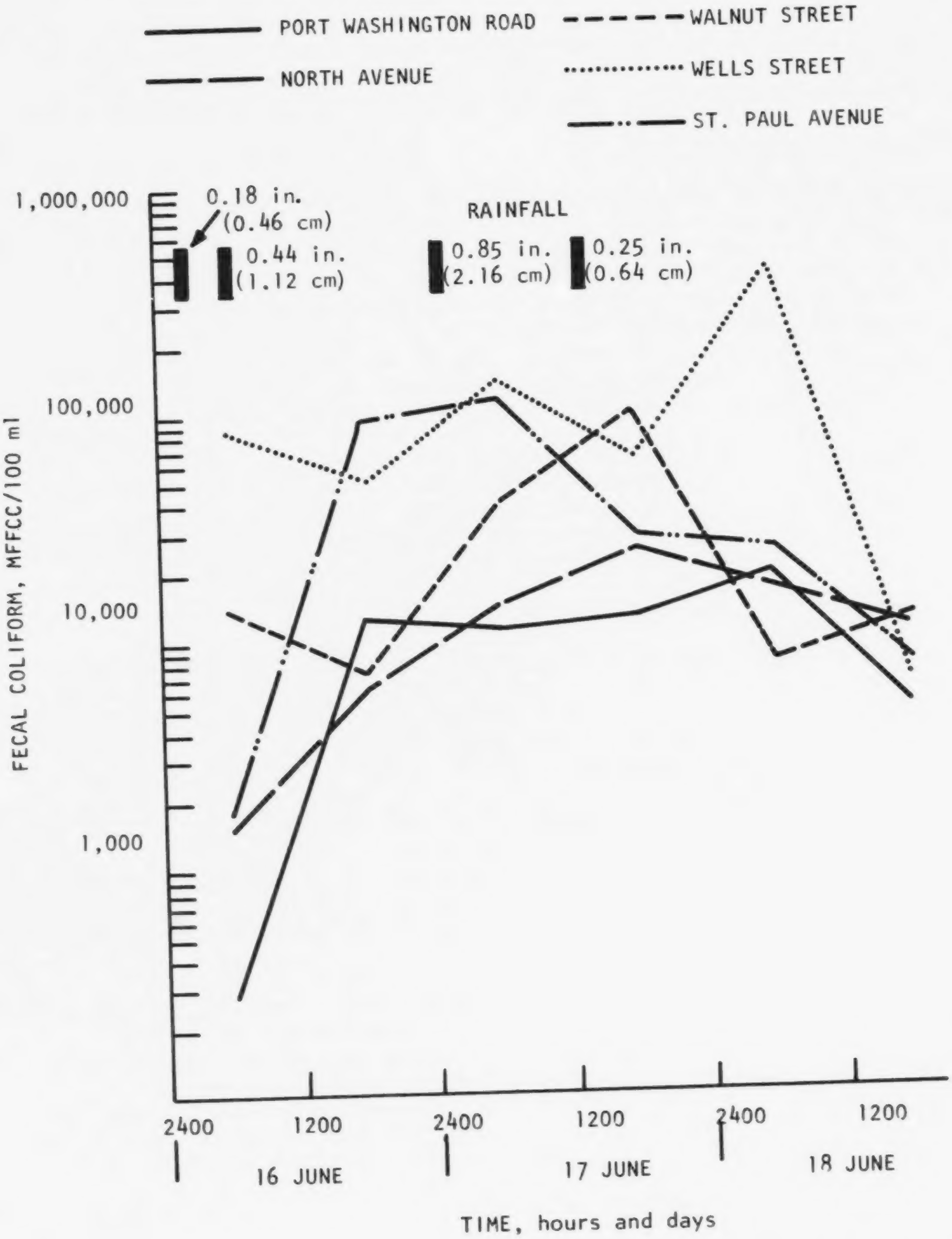


Figure 25. Monitored fecal coliform levels in the Milwaukee River during wet weather, June 16-18, 1978 (Survey 5).

survey. Instead of flushing out the high coliform levels, the high river flows continue to bring in significant levels of coliforms from upstream throughout the duration of the storm. Also, the rainfall through day 2 of the survey contributed to the high concentrations throughout the survey.

Survey 6 - Wet Weather Flow

The final intensive survey was conducted on the 26th through 29th of July, 1978. An average of 0.63 in. (1.60 cm) of rain fell during the period with the flow in the river peaking at 600 cfs (16.8 m^3/sec). The DO concentrations monitored during this survey are presented in Figure 26 which shows that there is very little change at a particular site during the three days of the survey. The trend of decreasing DO as distance downstream increases is again present during this survey. The DO approaches zero at the St. Paul Avenue site at the start of the second day of the survey, but rapidly increases to the 2.0 to 4.0 mg/l range shortly after.

The fecal coliform concentrations during this survey are graphically presented in Figure 27. The Port Washington site which is above the CSO area contains the lowest coliform levels while the St. Paul and Wells Street sites contain the highest. Very little change in concentrations is found between the start and end of the survey at a particular site.

CONTINUOUS DO AND TEMPERATURE MONITORING

Data from the continuous DO and temperature monitors located at the North Avenue Dam, Cherry Street and St. Paul Avenue was utilized to identify the impact of CSO on DO levels in the Milwaukee River. These monitors measured changes in DO at mid-channel and mid-depth in the river. From this data it was possible to examine three major trends in DO:

1. Dry weather variation prior to rainfall events.
2. Decline in DO following rainfall events.
3. Recovery of DO to dry weather levels.

It was also possible to contrast the changes in DO levels to the changes in river flow following rainfall events.

Figure 28 illustrates the changes in DO levels at St. Paul Avenue, and changes in flow at Estabrook Park and at the North Avenue Dam during August 2-7, 1977. This is one of the few periods during the study when the level recorder at the North Avenue Dam was operating properly. This figure shows data for a period which also includes a portion of Survey 4 (August 3-8, 1977). During the dry weather period on August 2 and 3, 1977 the DO levels ranged between 2.7 and 6.6 mg/l. Following rainfall, the DO declined 2.5 mg/l during a two hour period and then fluctuated around 1.0 mg/l until the next rainfall event on August 4, 1978. The DO then remained near 0.0 mg/l for nearly two days.

As expected the changes in river flow at the North Avenue Dam are slightly out of phase with the changes at Estabrook Park. There was an initial rapid increase in the flow rates, and then following rainfall on August 4

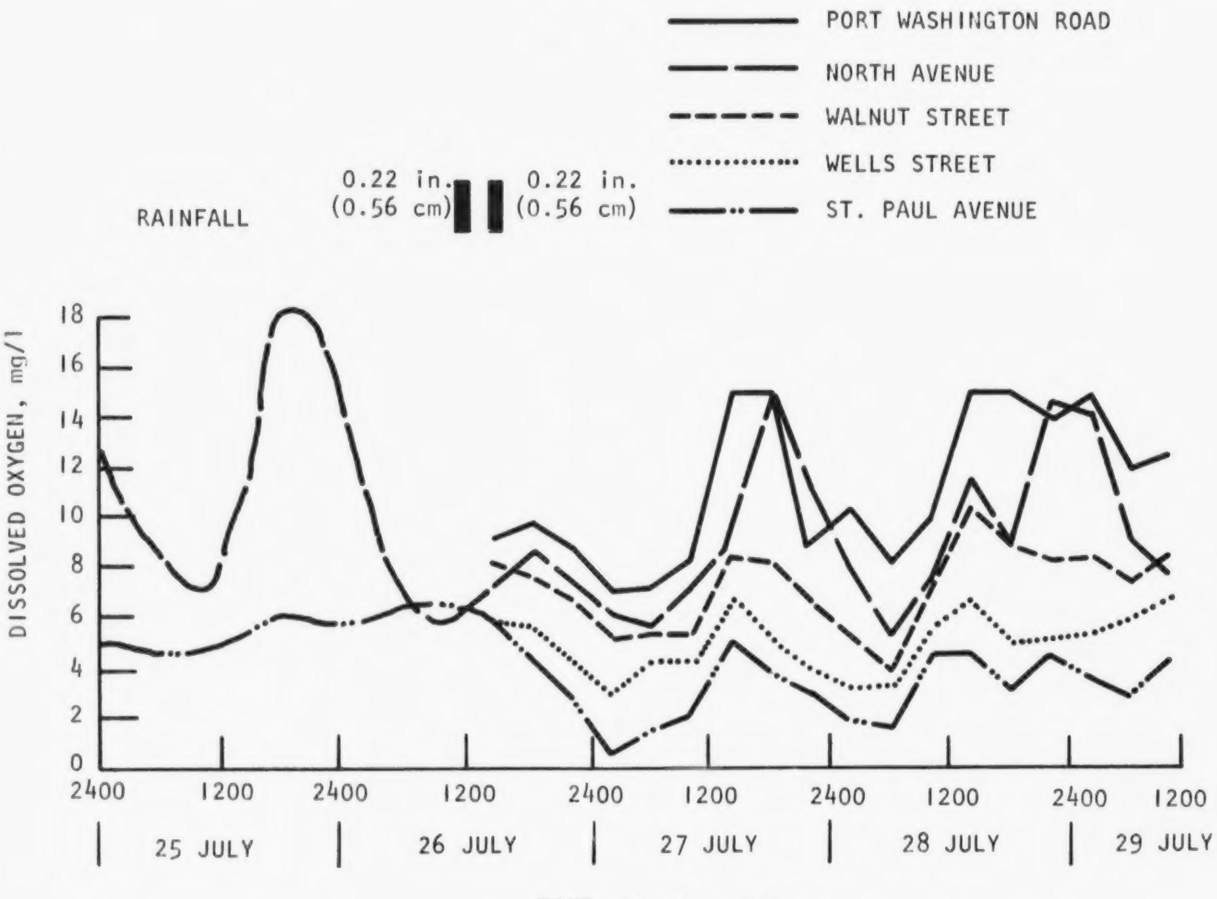


Figure 26. Monitored DO levels in the Milwaukee River during wet weather, July 25-29, 1978 (Survey 6).

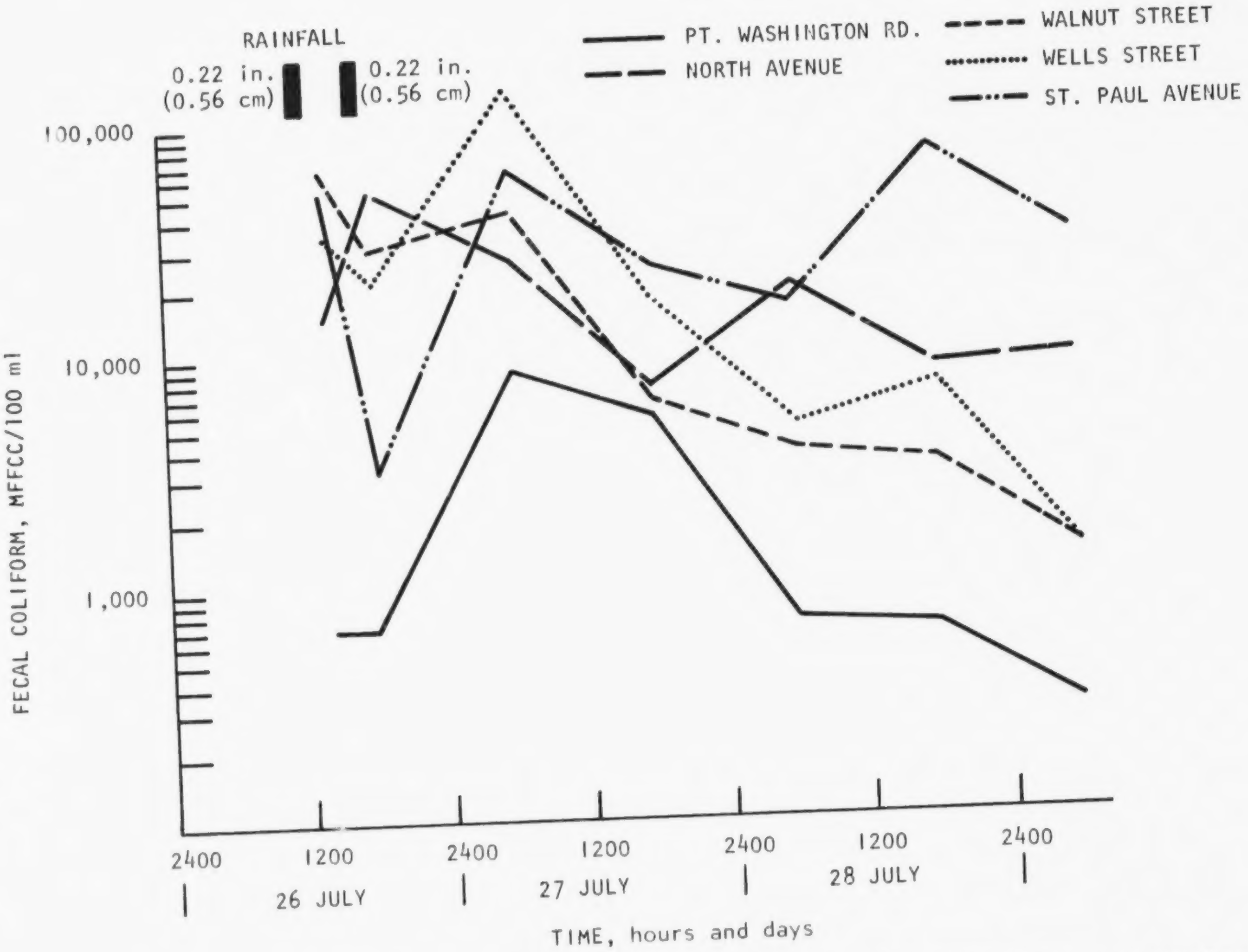


Figure 27. Monitored fecal coliform levels in the Milwaukee River during wet weather, July 26-29, 1978 (Survey 6).

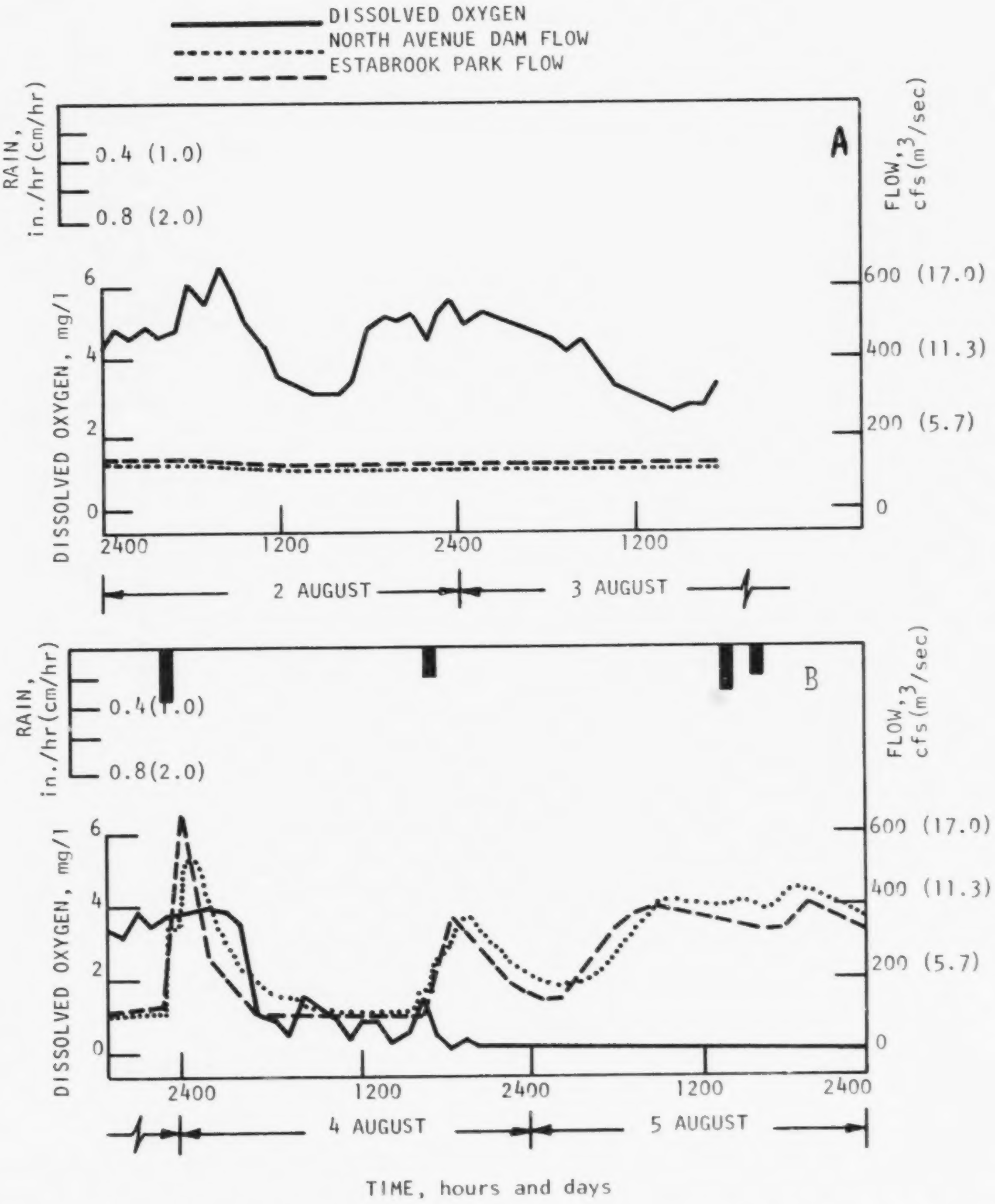


Figure 28. St. Paul Avenue continuous DO monitoring results for August 2-7, 1977.
a. Dry weather durations.
b. Wet weather DO variation.
c. DO recovery. (continued).

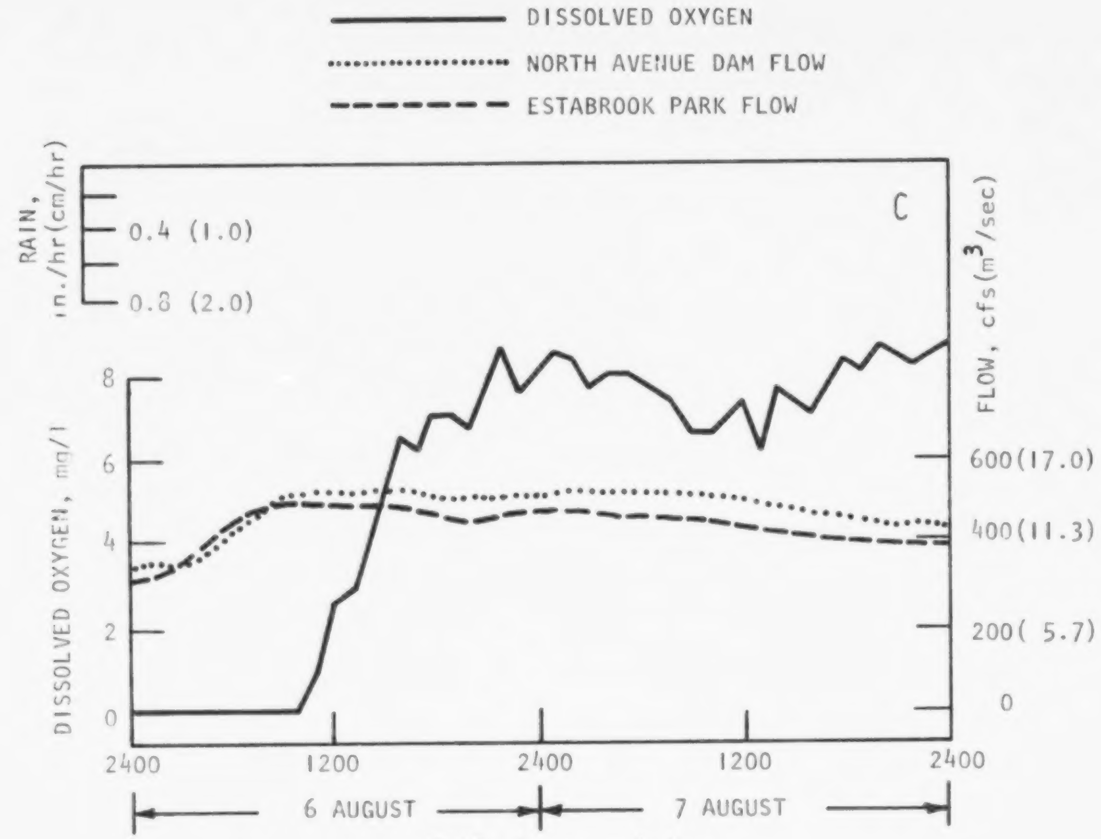


Figure 28 (continued).

and August 5, 1977 the flow increased to between 400 and 500 cfs (11.3 and 14.2 m^3/sec) at both of the flow monitoring sites. At a flow rate of 450 cfs (12.7 m^3/sec) the travel time from the North Avenue Dam to St. Paul Avenue is approximately 20 to 24 hours. The DO levels at St. Paul Avenue begin to recover approximately 24 hours after the flow at North Avenue reaches the 400 cfs (11.3 cu m/sec) level. The short duration increases in flow on August 4, 1977 appear to have little effect on DO in the lower river because the effect of these increases is dampered by the influence of Lake Michigan. When the flow increases to a constant level, the effect of the CSO is eventually flushed from the river and the DO at St. Paul Avenue recovers dramatically to nearly 8.0 mg/l.

Figure 29 illustrates changes in DO at St. Paul Avenue from July 26 to August 2, 1976. The first portion of this figure shows the dry weather changes of DO on July 26 and 27, 1976 followed by a gradual decline in DO after a small rainfall event. The effect of this event is very dramatic because the previous rainfall event on the CSO area was recorded on June 18, 1976. The river flow was also at an extremely low level of 85 cfs (2.4 m^3/sec) prior to this storm. On July 30, 1976 there was an extremely large storm of 1.60 in.

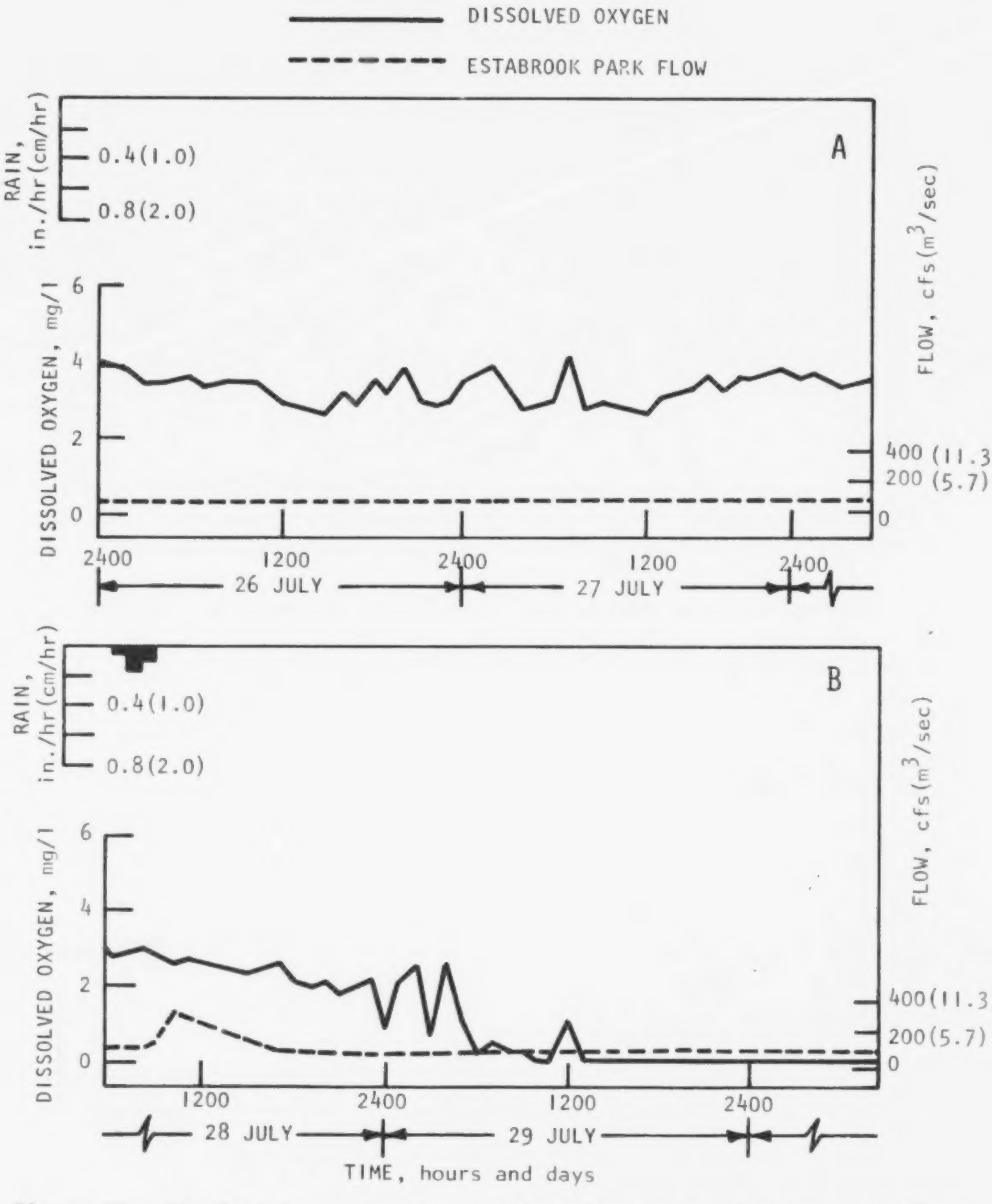


Figure 29. St. Paul Avenue continuous DO monitoring results for July 26 to August 2, 1976. a. Dry weather DO variations.
b.&c. Wet weather DO variations.
d. DO recovery. (continued).

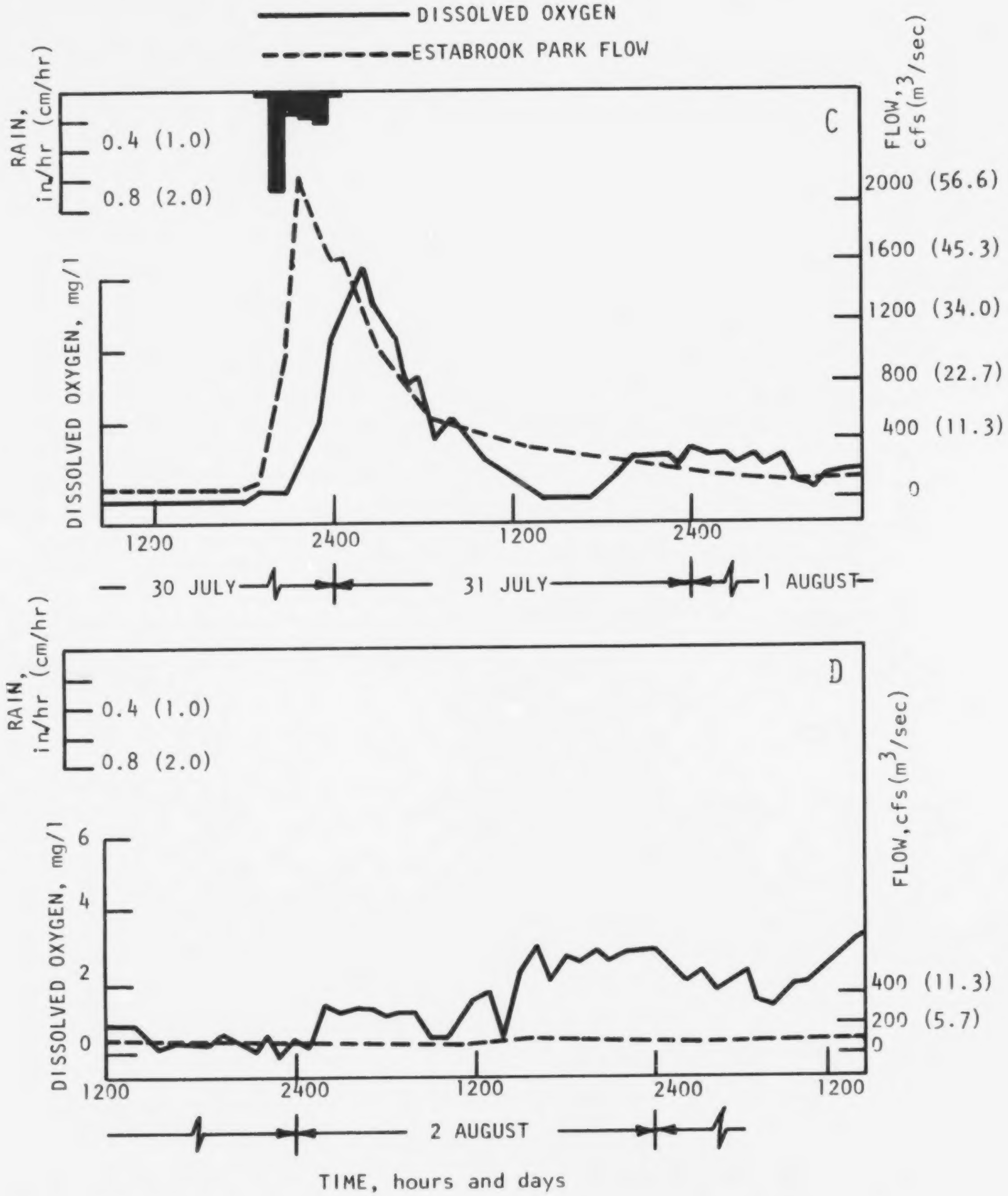


Figure 29 (continued).

(4.06 cm) of rain which caused a dramatic increase in flow at Estabrook Park to more than 2000 cfs (56.6 m^3/sec). This increase does effect the DO at St. Paul Avenue. The DO recovers for a short period of time due to the increase in flow and then drops rapidly back to 0.0 mg/l. The recovery of the DO back to dry weather levels occurs about 2 days later on August 2, 1976.

These were just two examples of the DO impact in the lower Milwaukee River following rainfall events as recorded with the continuous monitoring devices. Many other events equally as dramatic were recorded. The magnitude of the DO impact, duration of the depressed DO, rainfall and flow data was tabulated for several rainfall events. The analysis of this data is presented in Section 5.

SEDIMENT STUDIES

The DO conditions occurring within the lower portions of the river during the monitoring activities of this project have shown a wide range of values and types of response. In some overflow events the river has exhibited a dramatic loss of DO to near 0.0 mg/l and in other events the DO gradually decreases only a few mg/l and then recovers. The mechanism of this impact was thought to be related to the bottom sediments in the lake influenced portions of the river.

The purpose of the sediment investigations was to characterize these bottom materials as to their chemical and oxygen demanding potential. Once these parameters were known, further studies could be conducted to determine the mechanism of the rapid DO loss during selected overflow events. Following the identification of the mechanism of this impact, the source of these materials could be determined and related to the contribution from CSO. The monitoring of the sediment oxygen demand (SOD) was conducted to provide a means for the instream water quality model to duplicate the observed changes in DO during the intensive surveys. Further details of the modeling of the sediments will be discussed in Section 6, Modeling Studies.

Chemical Analyses

The sediments in the Milwaukee River were characterized during the summer of 1977 at four locations upstream of the CSO area contribution (Capitol Drive), three locations within the CSO area but upstream of the North Avenue Dam, and five locations in the lake influenced portion of the river. Sediment sampling consisted of collecting a sediment core of 20 in. (51 cm) in length at mid-channel and a grab sample at each of the quarter points of the river at each location. The sediment core was split in half in order to determine differences between the top portion of the sediment and the lower portions of the sediment. Cores could not be collected above the North Avenue Dam and at two lower river sites because the sediment deposits were not sufficiently deep.

Table A-15 in the Appendix lists the results of the chemical analyses performed on the sediment samples. There was no general trend in the differences in concentration between the top and bottom portion of the sediment cores. At one location, the top portion of the core had considerably higher concentrations of all parameters than the bottom portion of the core. However, at the other two locations where cores were collected, there was little difference between the top and bottom.

The variability in the concentrations of the parameters observed at the quarter points of a particular sampling location are likely due to differences in the water velocity, dredging, and the scouring and deposition effects of combined sewer outfalls. An excellent example of variability in the deposits in a cross section of the river is at the junction of the Milwaukee and Menomonee Rivers (Site I). The middle portion of the channel was dredged during the summer of 1977. No sample could be obtained from the eastern quarter point of the river because the substrate was extremely hard clay, and only a small grab sample could be obtained at mid-channel.

It is difficult to determine whether this lack of soft bottom material was due to dredging or the scouring effect of the flow from the Menomonee River entering the Milwaukee River. The western quarter point was located on the inside corner of the junction of the two rivers, therefore, the sediment contained more of the flocculant material that is characteristic of most of the samples from the lake influenced portions of the rivers. The mid-channel and western quarter point samples differed mainly in the moisture contents of the sediments and the concentrations of ammonia-nitrogen, total phosphorus and heavy metals.

Figures 30 to 32 illustrate the general trends in the sediment concentrations of chemical oxygen demand (COD), ammonia-nitrogen (NH_3-N), and lead (Pb) in the various reaches of the river. Figures A-1 to A-6 in the Appendix illustrate the observed trends in the other parameters. The values shown represent averages of the quarter point and core samples. In each of these figures there are obvious differences in the sediment concentration of the parameters in each reach of the river. Upstream of Capitol Drive, the ammonia-nitrogen values are approximately 10 percent of those observed between the North Avenue Dam and Capitol Drive. The physical characteristics of these reaches of the river are quite similar in terms of river velocity and sediment deposition rates. The only difference is the contribution of CSO beginning at Capitol Drive. Downstream of the North Avenue Dam, the sediment concentrations of ammonia-nitrogen were about twice those observed between Capitol Drive and the North Avenue Dam. This is likely the result of the slower river velocities with the subsequent build-up of more sediments. Also, the percentage of the CSO area contributing to the river increases almost linearly with distance downstream (see Figure 9). Similar differences in the sediment concentrations of COD and lead in the different reaches of the river can be identified in Figures 31 and 32.

Table 6 is a summary of the observed range of all of the measured parameters in the three reaches of the Milwaukee River. These ranges were developed from the raw data and not the averages presented in previous figures. For every parameter, without exception, the observed range below the North Avenue

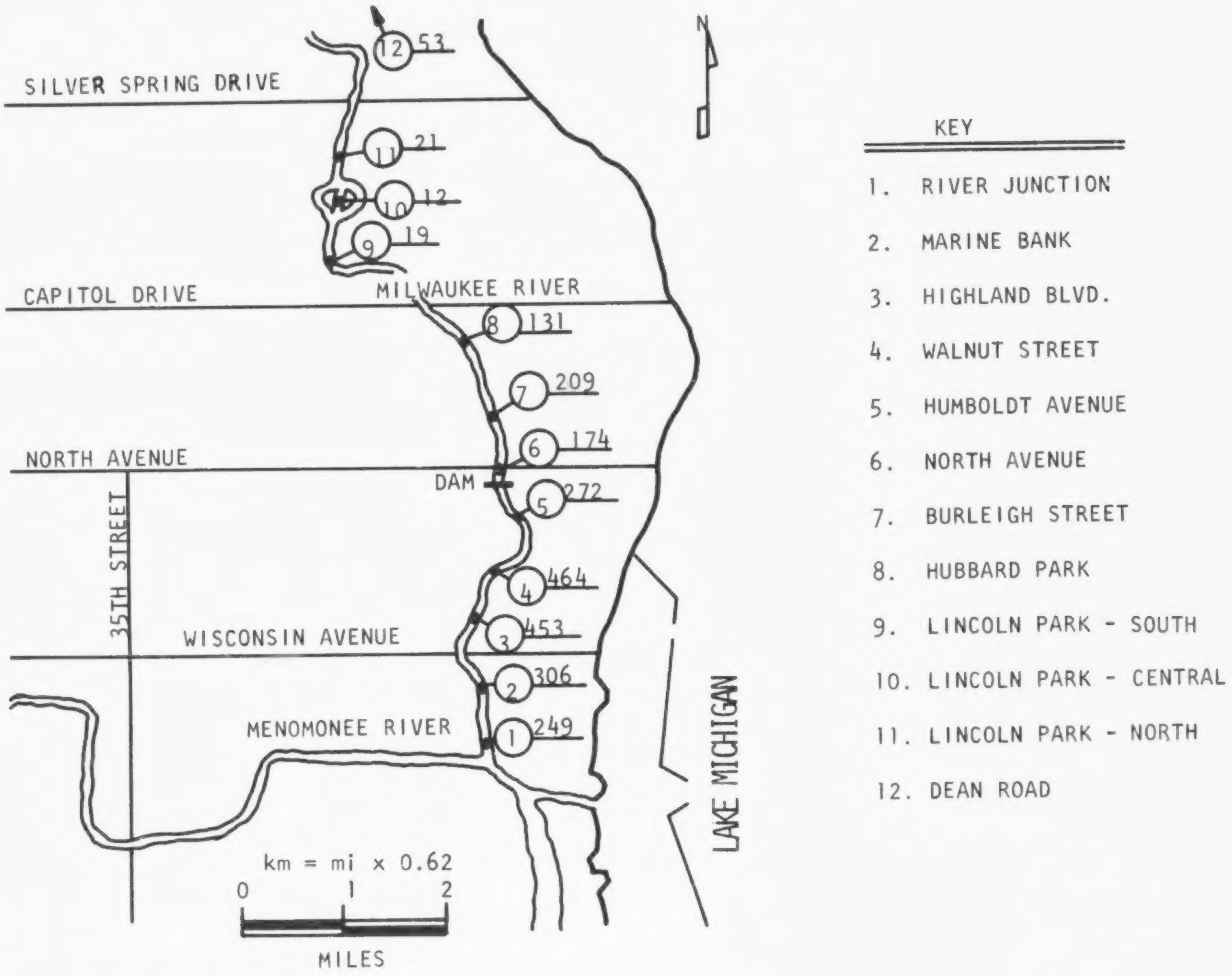

N
SILVER SPRING DRIVE
CAPITOL DRIVE
MILWAUKEE RIVER
NORTH AVENUE
DAM
35TH STREET
WISCONSIN AVENUE
MENOMONEE RIVER
LAKE MICHIGAN
km = mi x 0.62
0
1
2
MILES
12 53
11 21
10 12
9 19
8 131
7 209
6 174
5 272
4 464
3 453
2 306
1 249
KEY
1. RIVER JUNCTION
2. MARINE BANK
3. HIGHLAND BLVD.
4. WALNUT STREET
5. HUMBOLDT AVENUE
6. NORTH AVENUE
7. BURLEIGH STREET
8. HUBBARD PARK
9. LINCOLN PARK - SOUTH
10. LINCOLN PARK - CENTRAL
11. LINCOLN PARK - NORTH
12. DEAN ROAD

Figure 30. Ammonia-nitrogen (mg/kg) values in the sediments of the Milwaukee River.

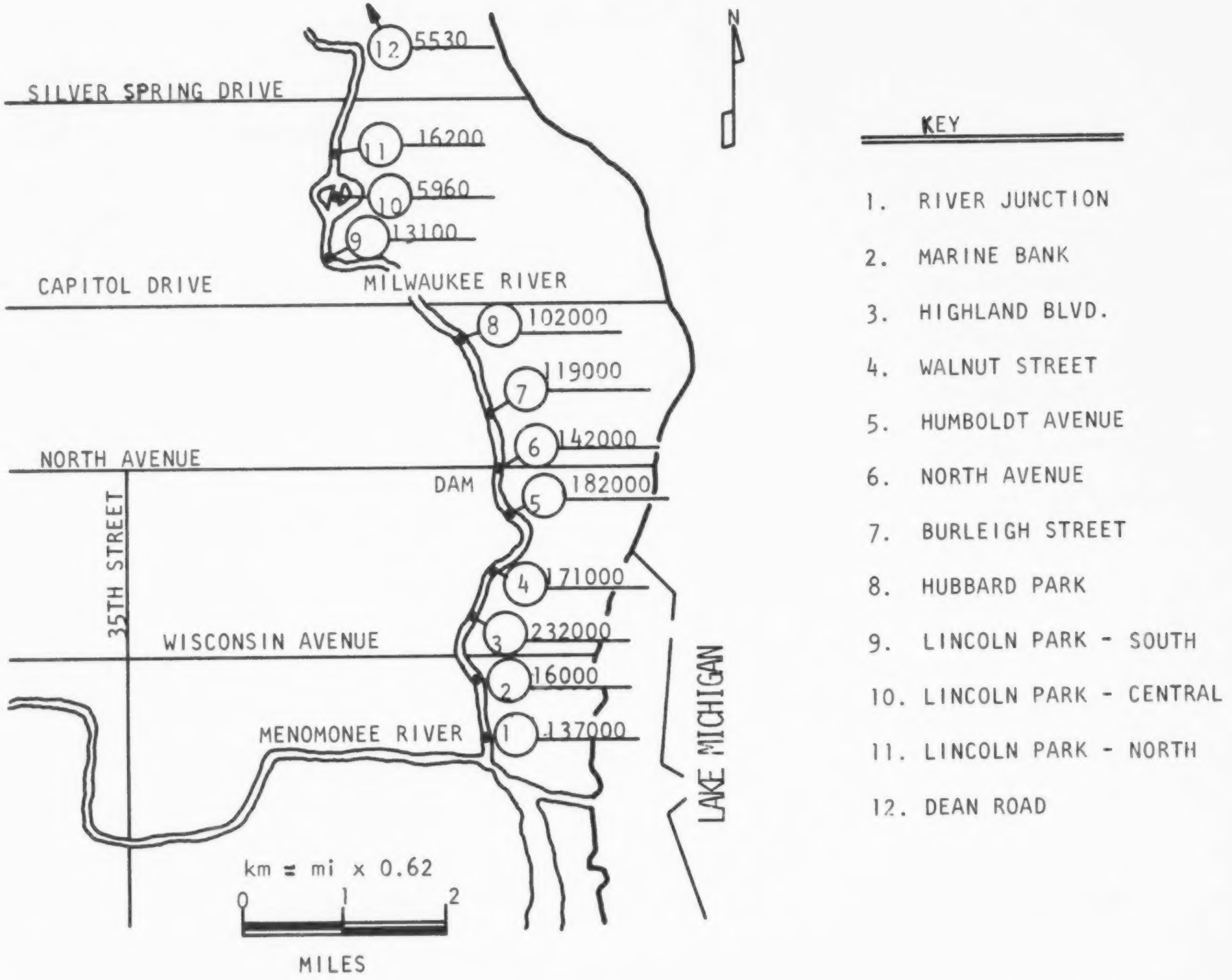

N
12 5530
SILVER SPRING DRIVE
11 16200
10 5960
9 13100
CAPITOL DRIVE
MILWAUKEE RIVER
8 102000
7 119000
6 142000
NORTH AVENUE
DAM
5 182000
35TH STREET
4 171000
3 232000
WISCONSIN AVENUE
2 16000
MENOMONEE RIVER
1 137000
LAKE MICHIGAN
km = mi x 0.62
0
1
2
MILES
KEY
1. RIVER JUNCTION
2. MARINE BANK
3. HIGHLAND BLVD.
4. WALNUT STREET
5. HUMBOLDT AVENUE
6. NORTH AVENUE
7. BURLEIGH STREET
8. HUBBARD PARK
9. LINCOLN PARK - SOUTH
10. LINCOLN PARK - CENTRAL
11. LINCOLN PARK - NORTH
12. DEAN ROAD

Figure 31. Chemical oxygen demand (mg/kg) in the sediments of the Milwaukee River.

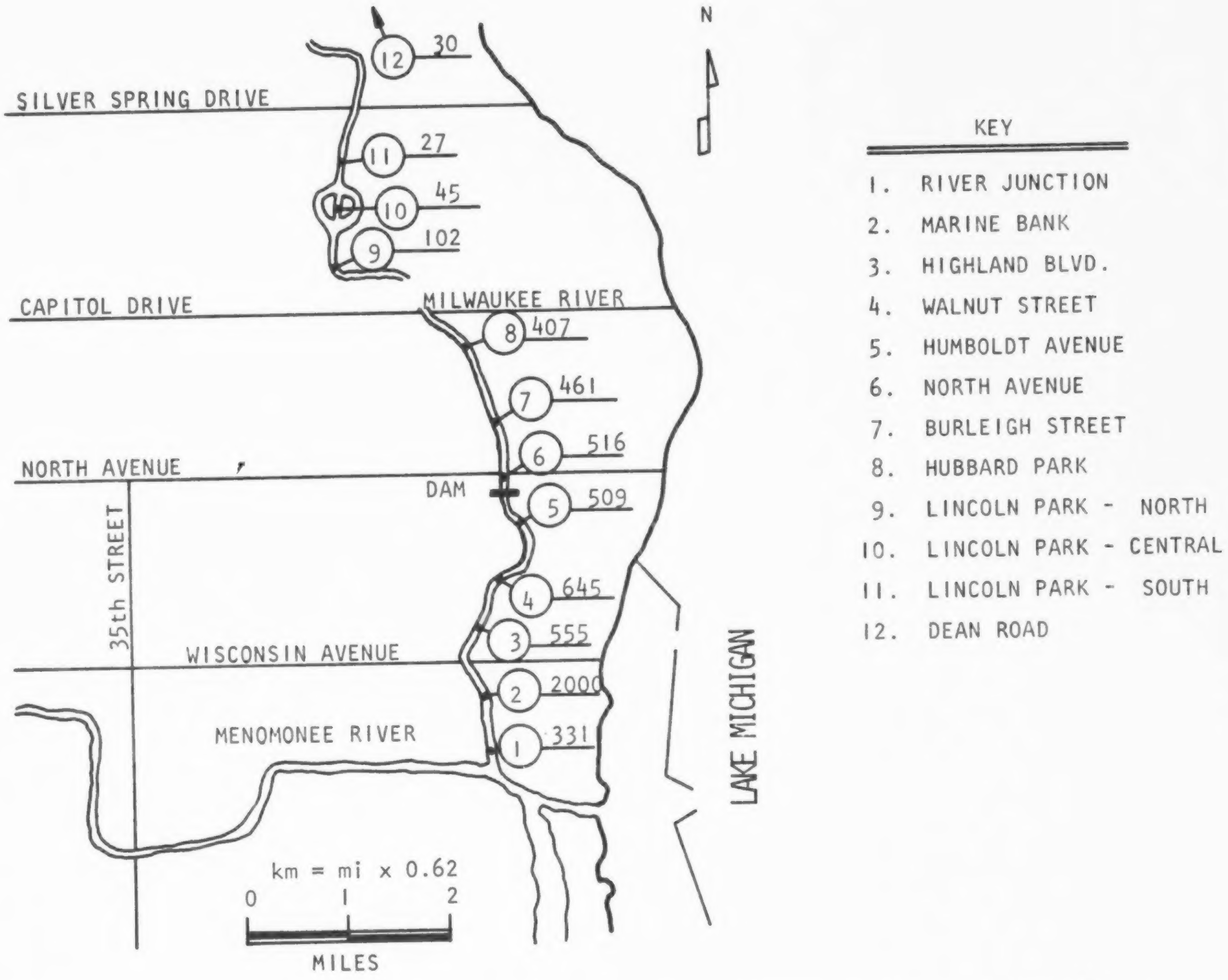

N
KEY
1. RIVER JUNCTION
2. MARINE BANK
3. HIGHLAND BLVD.
4. WALNUT STREET
5. HUMBOLDT AVENUE
6. NORTH AVENUE
7. BURLEIGH STREET
8. HUBBARD PARK
9. LINCOLN PARK - NORTH
10. LINCOLN PARK - CENTRAL
11. LINCOLN PARK - SOUTH
12. DEAN ROAD
12
30
SILVER SPRING DRIVE
11
27
10
45
9
102
CAPITOL DRIVE
MILWAUKEE RIVER
8
407
7
461
6
516
NORTH AVENUE
DAM
5
509
35th STREET
4
645
3
555
WISCONSIN AVENUE
2
2000
MENOMONEE RIVER
1
331
LAKE MICHIGAN
km = mi x 0.62
0
1
2
MILES

Figure 32. Lead (mg/kg) values in the sediments of the Milwaukee River.

TABLE 6. SUMMARY OF VARIABILITY OF CHEMICAL PARAMETERS DURING THE SUMMER OF 1977 IN DIFFERENT REACHES OF THE MILWAUKEE RIVER

	Below North Avenue Dam	North Avenue Dam to Capitol Drive	Upstream Capitol Drive
Volatile total solids (%)[1]	1.88-5.12	1.16-3.50	0.35-1.75
Chemical oxygen demand (mg/kg)[2]	85,983-380,000	8,820-195,000	577-35,186
Ammonia - N (mg/kg)[2]	60-743	20-334	9-85
Nitrite + nitrate-N (mg/kg)[2]	0.03-0.85	0.01-0.09	0.01-0.29
Total phosphorus (mg/kg)[2]	475-2,840	69-2,006	18-290
Iron (mg/kg)[2]	11,000-26,300	6,800-25,000	4,300-34,000
Cadmium (mg/kg)[2]	9-37	3-16	2-8
Zinc (mg/kg)[2]	53-826	110-492	21-171
Lead (mg/kg)[2]	34-6,350	49-668	24-239
Copper (mg/kg)[2]	21-528	3-141	2-36

[1] Values are percent of total sample weight as volatile total solids.
[2] Values are mg of measured parameter per kg of total solids.

Dam was considerably higher than in the other two sections of the river. The range of values for the reach between Capitol Drive and the North Avenue Dam generally overlapped the range observed downstream, however, the maximum values observed below the Dam were considerably higher than those upstream. Upstream of Capitol Drive the sediment concentrations of all the parameters except iron were considerably less than in the sections of the river receiving loadings from CSO.

The sediments in the Milwaukee River were also characterized during December 1976. Samples were collected at mid-channel only in the lower portion of the river. The results of the chemical analyses of these samples is listed in Table A-16 of the Appendix. These results are quite comparable to those observed during the summer of 1977. All of the measured parameters were in the same range during both surveys.

Three of the sediment samples collected during the winter were analyzed using a centrifuge in order to determine whether the pollutant parameters were associated with the solids or the interstitial water of the sediments. Samples collected at three sites on the Milwaukee River exhibit similar characteristics for centrifuge tests. These characteristics are the transfer properties of various pollutants from the sediments during the centrifuging process. The transfer properties are similar even though the moisture contents of the sediments are substantially different as shown in Table 7.

TABLE 7. MOISTURE CHARACTERISTICS OF MILWAUKEE RIVER SEDIMENTS

Sediment location	Total solids, % Uncentrifuged	Total solids, % Centrifuged	Volume of centrate, liters[1]
Harbor	16.7	48.5	3.93
St. Paul Avenue	29.1	50.1	1.44
Holton Street	34.0	54.9	0.73

[1] Volume of centrate per kg (dry weight) of centrifuged sediment.

The results in Table 8 indicate that nearly all parameters measured during the centrifuge tests remained with the solids of the sediments. Only ammonia-nitrogen appears to be readily soluble and thus transferred to the centrate. The heavy metals were expected to remain closely associated with the solids. However, the retention of BOD_{20}, COD and TOC with the solids was not expected as greater amounts of these parameters were thought to be soluble.

Inspection of the Table 8 values suggest that the sensitivity and accuracy of some tests may be inadequate for mass balance calculations. For example, the mass of TOC calculated to exist per kilogram of dry solids following the centrifugation of sediments exceeds the mass calculated to exist prior to centrifugation. In other words, the quantity of TOC contained in centrifuged sediments and centrate exceeds the quantity of TOC contained in the uncentrifuged sediments. Such generation of mass is not possible and is attributed to test procedure limitations. Regardless of such inconsistencies, the points made in the previous paragraphs remain valid due to the extremely small amounts of mass transferred to the centrate relative to the mass in the uncentrifuged sediments.

Settling Tests

Settling tests were conducted on Milwaukee River sediments suspended in river water in January, 1977 and February, 1978. The sediment samples in 1977 were from Wisconsin Avenue and the 1978 samples were collected at the junction of the Menomonee and Milwaukee Rivers. Settling tests were run on four different sediment/water concentrations as listed in Tables 9 and 10.

The settling curves for the sediments suspended in river water in 1977 (Appendix Figure A-7) indicates at least two phases of settling occur. Suspended solids concentrations rapidly decrease by at least an order of magnitude in the first hour of settling for the range of conditions conducted during the laboratory tests. Suspended sediment concentrations continued to decrease, but at steadily decreasing rates for the next 10 to 15

TABLE 8. SUMMARY OF MASS BALANCE CALCULATIONS FOR MILWAUKEE RIVER SEDIMENTS PRIOR TO AND FOLLOWING CENTRIFUGATION

	Weight of constituent (mg) per kg dry wt. of sediments								
	Harbor Sediments			St. Paul Sediments			Holton Sediments		
Parameter	Un-centrifuged	Centrifuged	Centrate	Un-centrifuged	Centrifuged	Centrate	Un-centrifuged	Centrifuged	Centrate
Total solids	1,000,000	996,900	3,070	1,000,000	998,800	1,160	1,000,000	999,300	630
Volatile solids	199,000	143,600	857	118,000	137,800	307	97,000	110,900	138
BOD_{20}	47,900	19,400	656	16,100	61,800	144	20,000	16,800	22
COD	260,800	687,160	487	160,000	1,009,800	180	152,000	226,800	45
TOC	80,800	223,000	173	61,900	260,500	82	64,700	252,100	23
Ammonia-N	4,200	2,690	1,180	269	40	137	590	470	45
Total-P	6,470	7,780	1.7	1,560	14,680	0.8	1,530	8,090	<0.1
Iron	24,000	-	21.6	18,000	-	12.4	18,000	-	2.3
Cadmium	39	-	0.4	10	-	<0.1	11	-	<0.1
Lead	285	-	0.4	405	-	<0.1	685	-	<0.1
Copper	165	-	0.3	111	--	<0.1	112	-	<0.1
Zinc	705	-	<0.1	410	-	<0.1	455	-	<0.1

hours. Settling of solids appeared to have ceased beyond 15 hours as solids concentrations are generally no longer decreasing.

TABLE 9. SETTLING CHARACTERISTICS OF MILWAUKEE RIVER SEDIMENTS FROM WISCONSIN AVENUE SUSPENDED IN RIVER WATER, JANUARY, 1977[1]

Sediment mixture[2], ml	Percent of initial suspended sediments having settling velocity (V) (in./hr) or greater			
	20	10	5	2
25	92	95	97	98
50	94	95	96	97
75	97	98	99	99
100	89	98	99	99

cm = 0.39 in.

[1] Estimated from settling plots.

[2] Volume of sediment added to river water for a total volume of one liter.

TABLE 10. SETTLING CHARACTERISTICS OF SEDIMENTS FROM JUNCTION OF MILWAUKEE AND MENOMONEE RIVERS SUSPENDED IN RIVER WATER, FEBRUARY, 1978[1]

Sediment mixture[2], ml	Percent of initial suspended sediments having settling velocity (V) (in./hr) or greater				
	20	10	5	2.5	1
25	87	88	93	94	98
50	86	92	94	97	99
75	90	93	95	98	99
100	82	94	96	98	99

cm = 0.39 in.

[1] Estimated from settling plots.

[2] Volume of sediment added to river water for a total volume of one liter.

These observations were illustrated by settling velocities calculated for each of the sediment and river water mixtures. In general, in excess of 90 percent of the sediments had settling velocities greater than 20 in./hr (51 cm/hr) (Table 9). The amount of suspended sediment that remained in suspension ranged from 1 to 3 percent of the original amount of sediments.

The settling tests were repeated in early 1978 using identical procedures. The settling curves for the settling tests conducted in 1978 also indicated at least two phases of settling (Figure A-8 in the Appendix). Similar to the 1977 test, rapid settling occurred for at least the first hour. Continually decreasing rates of settling occurred after approximately four hours of settling. However, contrary to the 1977 tests, settling of solids was continuing after 40 hours.

The settling rates of solids were slightly less for the 1978 tests compared with the 1977 settling tests. In general, greater than 80 percent of the sediments suspended in river water had settling rates exceeding 20 in./hr (51 cm/hr) for the 1978 tests (Table 10) compared with 90 percent for the 1977 test (Table 9).

Despite the slightly slower settling rates for the 1978 tests, the total percentage of solids settling within the test period was greater for the 1978 tests than for those conducted in 1977. Only 1 to 2 percent of the solids remained in suspension after 40 hours of settling during the 1978 tests. The differences in these settling test results are minor and are likely due to the different sediment sampling locations for the two tests.

Scouring velocities can be estimated from the settling data through two approaches presented by Fair, Geyer and Okun (19). One approach is to use the equation for scouring velocity based upon settling velocity:

$$V_d = (8/f)^{1/2} V_s$$

Where:

V_d = Scouring velocity.
V_s = Settling velocity.
f = Friction factor ranging from 0.02 to 0.04.

Using a settling velocity of 20 in./hr (51 cm/hr) the range of scouring velocity is 0.006 to 0.009 ft/sec (0.18 to 0.27 cm/sec).

A second estimate of scouring velocity is based upon the following equation (19):

$$V_d = [(8\ k/f)\ g\ (s-1)\ d]^{1/2}$$

Where:

k = 0.04 for sand and > 0.06 for sticky material.
g = 32.2 ft/sec^2 (9.8 m/sec^2).
s = Specific gravity of material.
d = Diameter of particle being settled.

To estimate an equivalent particle diameter, Stoke's Law is assumed to apply (i.e., laminar flow conditions and the settling particle is a sphere). Using a settling velocity of 20 in./hr (51 cm/hr) and specific gravity of particles of 1.05, the equivalent sphere diameter is approximately 0.01 cm. The particle with this diameter would have a scouring velocity of 0.08 ft/sec (2.4 cm/sec).

In summary, the scouring velocity range is approximately 0.01 to 0.1 ft/sec (0.30 to 3.0 cm/sec) for the particle having settling velocity of 20 in./hr (51 cm/hr). This range is calculated assuming the particle is nearly spherical in shape, that unhindered settling occurs and that laminar flow conditions prevail. Irregular shaped particles would have lower settling velocities, which would result in lower scouring velocities than previously estimated. Thus, at least 10 to 20 percent of the Milwaukee River sediments have scouring velocities lower than 0.1 ft/sec (3.0 cm/sec). During summer conditions the scouring velocity is probably somewhat faster than indicated by these tests. The sediment samples for these tests were obtained during the winter months when the river flow was less than 100 cfs (2.8 m^3/sec) and the river velocities were quite slow. Because of these slow velocities, the sediments probably contained a higher percentage of the particles with low scouring velocities than found during higher flow conditions in the spring and summer. These tests still provide a relative indication of the velocity required to scour sediments in the lower Milwaukee River.

Sediment Oxygen Demand Tests

In order to determine the significance of the sediments on the dissolved oxygen balance in the Milwaukee River, both *in situ* measurements and laboratory testing was conducted. The *in situ* technique for measuring sediment oxygen demand (SOD) rates was developed by Lucas and Thomas (20). The respirometer consisted of a plexiglass chamber with a DO probe attached inside and a small 12 v submersible pump attached outside for circulation of water (Figure 33). The SOD rates were determined from the decline in DO over time after the chamber was sealed on the sediment surface. The *in situ* SOD rate was calculated using the following formula:

$$SOD = \frac{(C_i - C_f)V}{tA}$$

Where:

SOD = Sediment uptake rate in gm O_2/m^2-day.
V = Volume of confined water in m^3.
A = Bottom area within chamber in m^2.
t = Test period in days.
C_i = Initial measured DO in chamber in mg/l.
C_f = Final measured DO in chamber in mg/l.

This formula provides the SOD rate on an areal basis and can be converted to a volumetric rate in the river by division by the mean depth at the sampling location.

The SOD rate was determined twice at five locations in the lower Milwaukee River and once at three locations upstream of the North Avenue Dam. The measured SOD rates are listed in Table 11. These rates have been corrected for temperature differences using a Van't Hoff-Arrhenius expression as determined by McDonnell and Hall (21). The two rate measurements conducted at each location were generally quite close despite being taken at intervals up to one and one-half months apart. There was a substantial difference in the observations only at Humboldt Avenue. This may have been due to the variability in the sediment depth at this site which is in close proximity to the North Avenue Dam. Except for the Walnut Street site, there appears to be a general trend of increasing SOD with distance downstream. The SOD

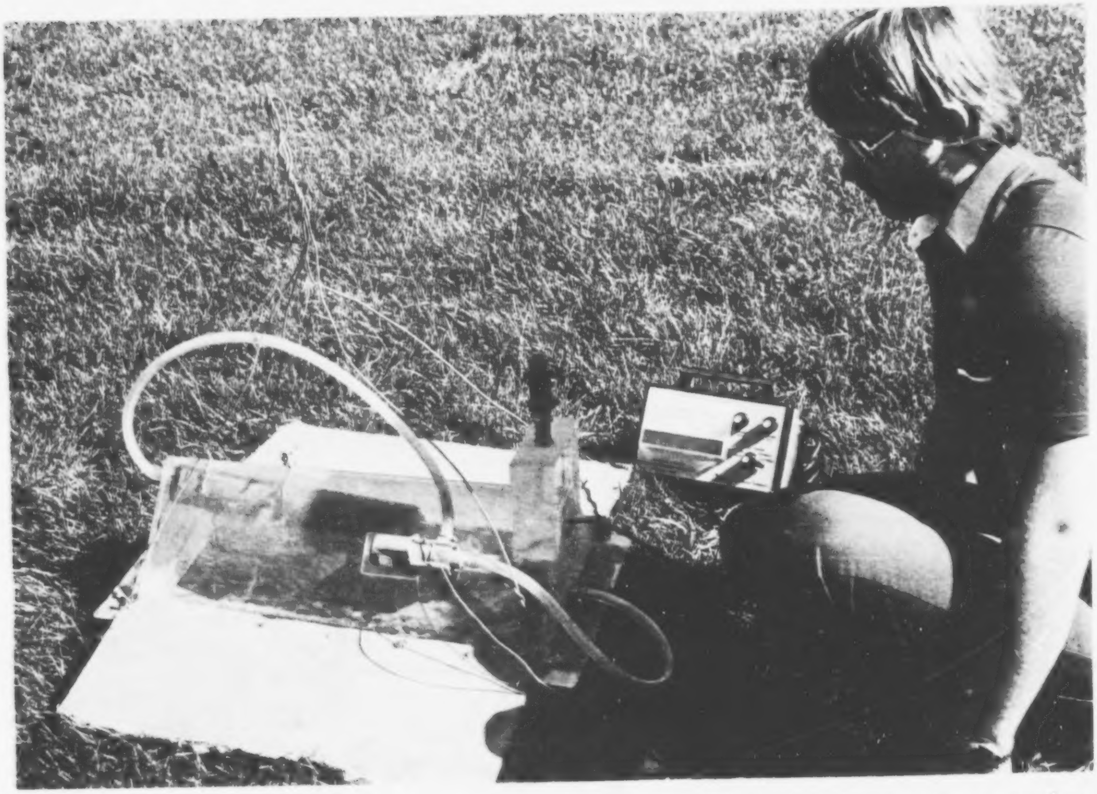


Figure 33. Photograph of sediment oxygen demand respirometer for measuring *in situ* SOD rates.

TABLE II. VARIATION OF *IN SITU* SEDIMENT OXYGEN DEMAND RATES IN THE MILWAUKEE RIVER.

Location	Distance upstream[1], km	Date	SOD gm/m^2-day
River Junction	0.0	7/8/77 8/1/77	6.6 6.7
St. Paul Avenue	0.3	6/23/77 8/1/77	5.3 5.5
Highland Blvd.	1.6	6/27/77 8/1/77	3.7 4.3
Walnut Street	2.3	6/27/77 8/1/77	6.1 6.7
Humboldt Avenue	3.4	6/27/77 8/1/77	3.7 1.8
Burleigh Street	5.6	7/14/77	4.5
Lincoln Park	11.8	7/14/77	3.0
Dean Road	20.9	7/14/77	0.0

[1] The distance upstream is listed from the junction of the Milwaukee and Menomonee Rivers.

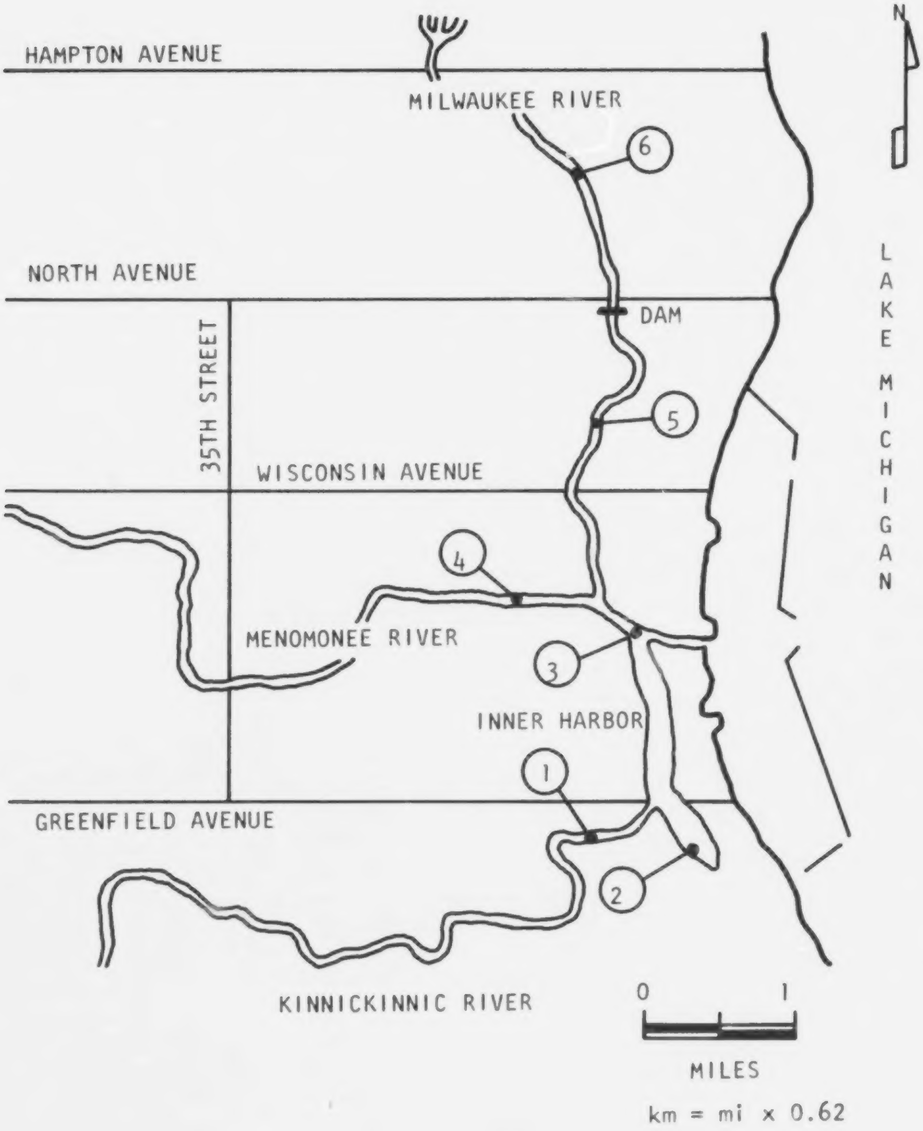
rates measured at Walnut Street were in the same range as those observed at the junction of the Menomonee and Milwaukee Rivers.

Laboratory measurements of the undisturbed and disturbed SOD rates in the river were conducted in a plexiglass cylinder lined with aluminum foil to avoid light penetration. The cylinder was 8.5 in. (21 cm) in diameter and 4.2 ft (1.3 m) in height. A two liter sediment sample was placed in the bottom of the cylinder and overlaid with 21 liters of river water. This system was allowed to settle and then was aerated with compressed air in order to saturate the water with oxygen. An air stone was utilized to diffuse the air to avoid disturbance of the sediments. DO readings were taken intermittently at mid-depth in the cylinder with a portable DO probe and meter. This continued for a five day period while the system remained quiescent. The decline in DO during this period was used as a verification of the SOD rates determined with the in situ technique. Following the measurement of the undisturbed SOD, the system was aerated again. The decline in DO was then measured as the sediments were disturbed by circulating the river water with a centrifugal pump. The SOD rates measured under these conditions were utilized as a good approximation of the demand exerted in the Milwaukee River when the sediments are scoured.

Sediment samples for laboratory SOD rate determination were collected in each of the three rivers entering Lake Michigan and the inner harbor as shown in Figure 34 and Table 12. The undisturbed rates measured in the laboratory are considerably smaller than those measured in situ. This is likely due to oxidation of the sediments during sampling, storage, and set up of the laboratory system. The aeration of the system is also a likely cause of the reduced undisturbed values.

The two sites in or near the inner harbor (Sites 2 and 3) had significantly larger disturbed SOD values than the other sites on the three rivers. The lower values obtained further upstream in each of the three rivers were quite similar. All three rivers contribute to the sediments in the inner harbor, therefore, it is feasible that the largest disturbed SOD values were obtained from the sample collected in this area. There did not seem to be any relationship between the undisturbed SOD and the disturbed SOD. The site with the second largest undisturbed SOD (Site 4) had one of the smallest disturbed SOD rates.

In order to determine whether the density of the sediments affected the SOD measured, the rates were also calculated on a weight basis (gm O_2/gm sediment-day). Since all of the sediment samples were approximately the same density and of similar moisture content, this change in units had little effect on the previously mentioned trends in the SOD values in the rivers. The areal expression of SOD values is the conventional form for these rates.


HAMPTON AVENUE
MILWAUKEE RIVER
6
NORTH AVENUE
DAM
35TH STREET
5
WISCONSIN AVENUE
4
MENOMONEE RIVER
3
INNER HARBOR
1
GREENFIELD AVENUE
2
KINNICKINNIC RIVER
LAKE MICHIGAN
N
0
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MILES
km = mi x 0.62

Figure 34. Sediment sampling locations for bench scale sediment oxygen demand characteristics.

TABLE 12. BENCH SCALE DETERMINATIONS OF SEDIMENT OXYGEN DEMAND UNDER UNDISTURBED AND DISTURBED CONDITIONS[1].

Map location no.	Description	Undisturbed SOD, gm/m^2-day	Disturbed SOD, gm/m^2-day
1	Kinnickinnic River at First Street	0.65	430
2	Mooring Basin in Inner Harbor	1.40	1,370
3	Milwaukee River at RR Bridge	2.10	800
4	Menomonee River at Great Lakes Coal	1.70	270
5	Milwaukee River at Highland Avenue	1.40	360
6	Milwaukee River at Hubbard Park	0.33	66

[1] The laboratory sediment oxygen demand determinations were carried out at a temperature of 20 ± 0.5 C.

The results of the sediment oxygen demand tests indicate that the SOD rates (as determined by in situ measurements) in the lower Milwaukee River range between 1.8 and 6.7 gm/m^2-day and the average SOD rate for this portion of the river was 5.0 gm/m^2-day. The laboratory measurements of the SOD rates indicate that the oxygen demand of the sediments in the rivers and inner harbor in Milwaukee may exceed 1000 gm/m^2-day when the sediments are disturbed or agitated. This represents more than a 100 fold increase in the SOD rate. The significance of these in situ and disturbed SOD rates on the DO balance of the Milwaukee River is discussed in Section 5 of this report.

SECTION 5

ANALYSIS AND DISCUSSION

The field data collected as part of this project was used in this section to define the source and the mechanism of the change in water quality following overflow events. A methodology was developed to predict the DO impact and input to a receiving water model. Therefore, the relationships between the various elements and sources of the water quality impact could be evaluated in the following sections of this report.

HYDRAULIC STUDIES

In addition to providing data on travel times within the river during different flow conditions, the results of the hydraulic studies have further implications. The most significant finding was that the velocities in the lower portions of the river were not fast enough to cause the scouring of the bottom sediments during low flow conditions. During river flows of approximately 200 cfs (5.7 m^3/sec), the measured velocity between Walnut Street and St. Paul Avenue was just 0.06 ft/sec (0.018 m/sec). This velocity was in the range where slight sediment scouring could occur as determined with the sediment settling tests. However, the scouring velocities determined in the laboratory must be considered rough estimates and the velocities near the bottom sediments were much lower than the average river velocity. The results of the velocity monitoring near St. Paul Avenue indicated that velocities were not only quite low near the sediments, but the direction was quite variable. Some of the velocities measured near the river bottom during flows of 100 cfs (2.8 m^3/sec) and flows reaching 450 cfs (12.7 m^3/sec) were sufficient to cause some sediment scouring. However, the oscillating nature of the direction of the measured velocities indicated that there was little net downstream velocity near the sediments.

The trends in river velocity are important to the analyses which follow since they indicate that the scour of bottom sediments in these portions of the river does not occur primarily as a function of instream flows. The continuous DO recorders in these reaches have shown that rapid instream changes in flow have little effect on the downstream DO conditions. This fact was tested in early Spring of 1977, when the flood control dam above Estabrook Park was opened to allow the spring snowmelt flows to be released to the downstream reaches. Although the flows in the river increased by more than 300 cfs (8.5 cu m/sec), the DO record downstream indicated no change as a function of sediment scour. This is reasonable since the estimated change in the mean velocity in the lower river due to this change in flow is only 0.10 ft/sec (0.03 m/sec). The increase in the velocity near the sediments would be much smaller.

A second finding of the hydraulic studies that has provided insight into the mechanism of the water quality impacts during wet weather is the dispersion characteristics of the CSO outfalls. The dye injection at a CSO that occurred during low CSO discharge flows tended to remain at the top portion of the instream water column and the dye also remained on the same side of the river that the outfall was situated. This characteristic is important to remember since the submerged outfalls do not cause rapid mixing actions within the river during low CSO discharge flow conditions. When the outfall was flowing at higher velocities, then the dye plume was well dispersed across the river almost instantaneously after the injection. This rapid dispersion was thought to be the mechanism through which the bottom materials were scoured when the instream velocities were insufficient to affect this change. The data on velocities in the river generated from CSO discharges support this theory. This will be discussed in more detail in relation to the sediment investigations.

SEDIMENT INVESTIGATION RESULTS

The data generated within the sediment investigations portion of this project are the most important to deciphering the contribution of CSO to the DO impacts of the Milwaukee River. The first finding of note in these investigations is the vast differences in the upstream versus downstream portions of the river. For example, the ammonia-nitrogen concentrations of the sediments above the CSO area are significantly less than the concentrations in the CSO influenced portions of the river. As the distance downstream increases, the area of CSO contributing to the river also increases. Because of this, the ammonia-nitrogen concentrations below the North Avenue Dam were approximately double the concentration in the CSO influenced area above the dam. This is an indication of the influence of CSO on the bottom materials from a chemical characteristic basis. The same trend can be found in the SOD rate measurements taken in these areas. The general trend of these measurements was an increase in SOD with distance downstream.

The data must be examined to determine the significance of SOD on DO levels during dry weather and during wet weather conditions. Even during the dry weather survey (Survey 1) the DO in the lower portions of the river, particularly at St. Paul Avenue, approaches the 2.0 mg/l DO standard. This was partially due to the low flow conditions during this survey when the average flow was 74 cfs (2.1 m^3/sec). The significance of the dry weather or undisturbed SOD can be illustrated with the following hypothetical case. For example, if the river flow is 100 cfs (2.8 m^3/sec) and the SOD is taken as an average of the measured values, the SOD can account for a decline in DO of approximately 4.4 mg/l between the North Avenue Dam and the junction of the Milwaukee and Menomonee Rivers. Calculated on a volumetric basis, this means a rate of 0.04 mg O_2/l-hr in the lower portions of the river. Likewise, if the ultimate BOD in the river is assumed to be 5.0 mg/l as estimated with data from the dry weather survey, the soluble load can account for a decline in DO between the same locations of approximately 2.1 mg/l. This is assuming a BOD reaction rate coefficient of 0.1 per day as utilized in the receiving water model for this portion of the river. The volumetric rate of this soluble demand is approximately 0.02 mg O_2/l-hr. This indicates that the SOD is twice as significant during dry weather as the soluble demand.

The relationship of SOD to the DO conditions within the river was not fully understood until the laboratory studies of the undisturbed and disturbed demand were completed. These studies indicated that the potential exists in the sediments to cause the rapid decline in DO that had been observed with the continuous DO monitors. In addition, although the downstream reaches were low in DO during dry weather, they still remained above the 2.0 mg/l standard for these areas. If SOD was the source of the rapid DO sags, then the undisturbed demand could not account for this mechanism. Implementation of the undisturbed SOD rates into the receiving water quality model only served to lower the DO values in the river in both dry and wet weather conditions. The mechanism by which the sediments were disturbed and how this mechanism could be implemented in the water quality model led to the investigation of velocities at the bottom of the river during wet weather events.

The scour of bottom materials within the river by discharges from submerged CSO outfalls was verified with the velocity studies and with visual observations. The velocities measured in the vicinity of the submerged outfalls were extremely large and capable of scouring considerable quantities of sediment. The effect of the scouring can be illustrated with the rainfall, DO and CSO velocity data for a storm occurring on August 3, 1977. Figure 35 depicts the response of the river at St. Paul Avenue in terms of DO to this rainfall event and the subsequent scouring of sediments due to CSO discharges. An instream CSO velocity of nearly 12.0 ft/sec (3.7 m/sec) was measured 1.0 ft (0.3 m) above the sediment surface at a distance of approximately 30 ft (9.1 m) from a submerged CSO outfall at St. Paul Avenue. This velocity was measured just after discharges from this CSO outfall began and the direction of the velocity was perpendicular to the river. After discharge from the outfall ceased, the river velocity was variable in direction and very small in magnitude. The rapid decline of DO shown in Figure 35 following the rainfall event is the result of the scouring or agitation of bottom sediments throughout the lower Milwaukee River from discharges from submerged CSO outfalls. This CSO velocity measurement, as well as other measurements indicate that sediment scouring from submerged CSO discharges does occur in the lower portion of the river.

Visual observations in the lower river have also indicated the significance of sediment scouring. During severe storm events, the plumes from the numerous outfalls and the resulting scour of sediments were observed. The overall significance of scouring from CSO discharges is evident when one considers that there are approximately 40 outfalls discharging into the Milwaukee River below the North Avenue Dam. Although some of these outfalls have irregular shapes, the majority are circular and range from 36 in. (0.9 m) to 96 in. (2.7 m) in diameter. Also most of them are submerged. Averaging the number of outfalls evenly over the length of this portion of the river, approximately one CSO outfall exists every 300 ft (95 m) of river. Considering that many of the outfalls enter the river in pairs from opposite banks, and considering frequency of the outfall occurrence, the scouring potential from these outfalls can be quite significant.

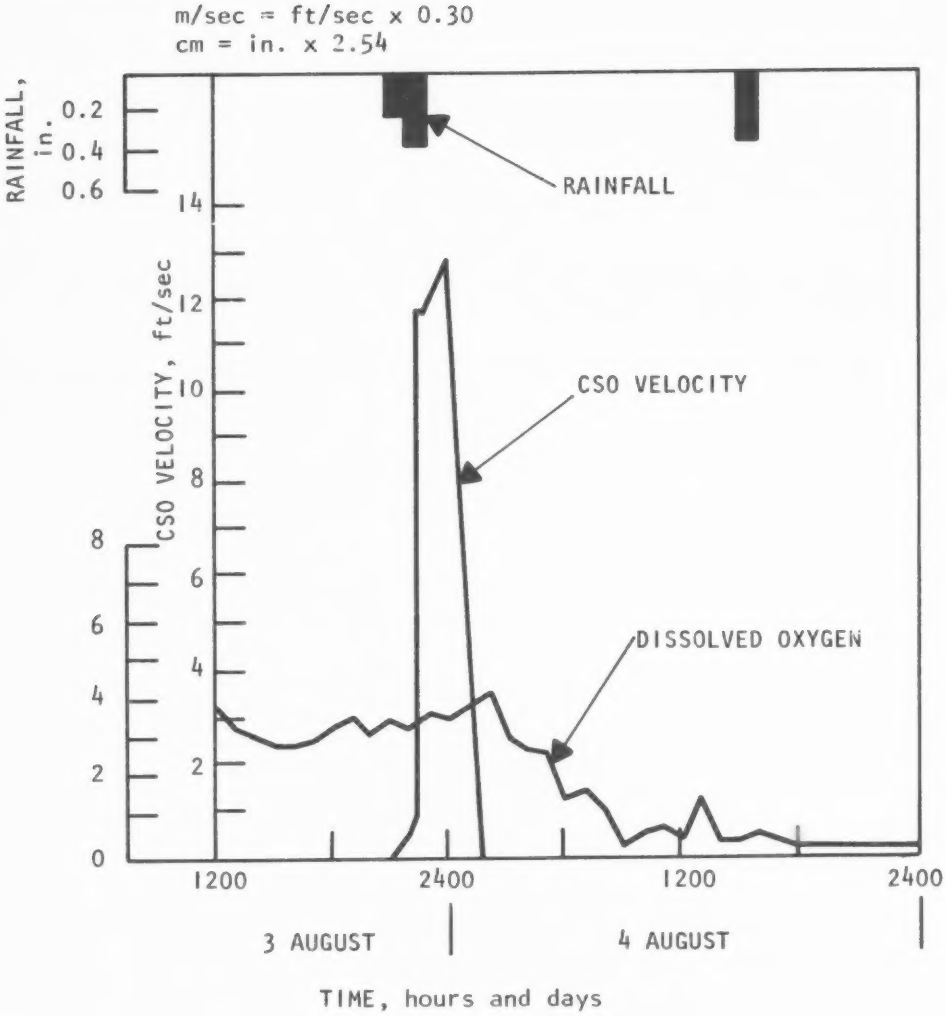


Figure 35. The observed decline in DO, instream velocity from a submerged CSO outfall, and rainfall volume at St. Paul Avenue in the lower Milwaukee River on August 3-4, 1977.

PREDICTION OF THE DO IMPACT

Due to the severe impact which sediment scouring has on DO levels following CSO events, the data collected during the study had to be analyzed in order to develop a consistant methodology to adjust SOD parameters for the calibration of Harper's water quality model. For this methodology an empirical expression was developed utilizing a forward (stepwise) multiple regression program. Data on DO impact from the continuous monitoring devices, rainfall and river flow were used for this analysis. Two different parameters were utilized as the dependent variables. The first was the change in DO (ΔDO) expressed in $gm\text{-}O_2/m^2$ at a continuous monitoring site following a rainfall event. The areal expression for ΔDO was utilized so that it could be easily related to the SOD rate which also was expressed on an areal basis. The other parameter was the duration of the DO decline (T) in hours.

Only the data on DO from the St. Paul Avenue continuous monitoring site was utilized for this analysis for two major reasons:

1. The most severe impact on DO levels was generally observed at this site.
2. If the impact could be predicted at this site, the decline in DO at other sites could be predicted by assuming that the magnitude of the wet weather demand increased with distance downstream from the North Avenue Dam.

The major assumption in the development of this procedure for predicting the DO impact is that the majority of the observed DO decline is due to sediment scouring. Some of the decline in DO is due to the organic loading from CSO, storm sewers and upstream loads. However, the rates of DO decline far exceed the rates which could be due to the measured BOD_5 and TOC concentrations in the river. Previous modeling studies have also been unable to account for the large wet weather demand (11, 12). In order to minimize correlating the decline in DO due to the soluble oxygen demanding materials, only the rapid decline portion of the DO curves were utilized in determining the ΔDO values. Figure 36 illustrates the methodology used to obtain these values.

Several independent variables were selected for the regression analysis. These variables were as follows:

1. Rainfall volume.
2. Average rainfall intensity.
3. Peak rainfall intensity.
4. CSO volume.
5. Peak river flow.
6. Change in river flow.
7. Antecedent dry period.
8. Natural log of the antecedent dry period.

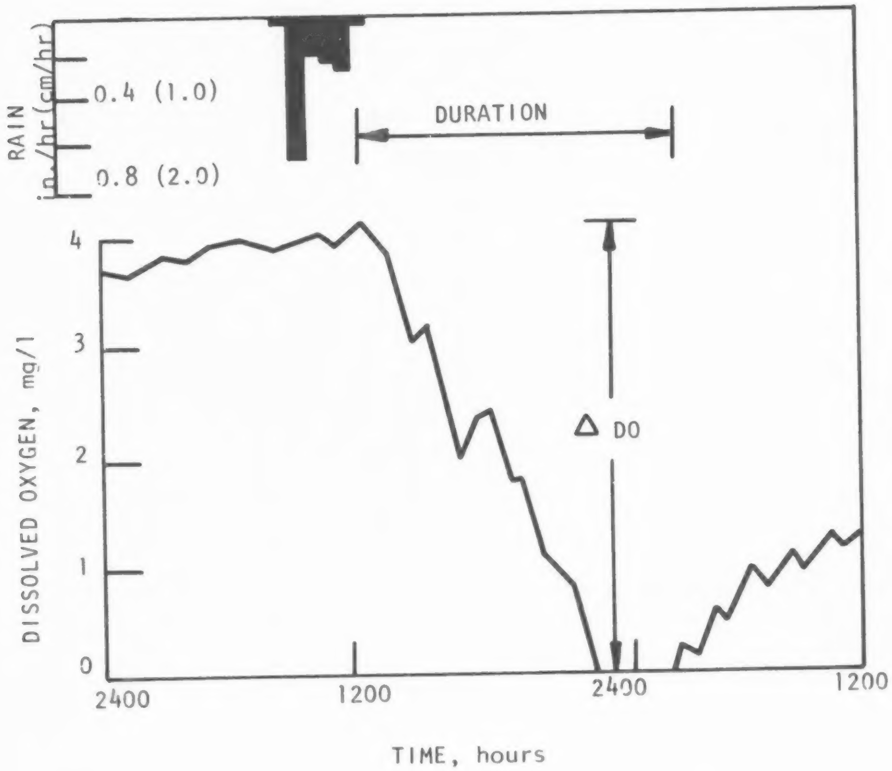


Figure 36. Methodology utilized to obtain Δ DO and duration values from continuous DO records at St. Paul Avenue.

The rainfall data used was from the Broadway Street precipitation gauge (Figure 2) which is located in the drainage basin tributary to the lower portions of the river. The CSO volume was estimated using the calibrated STORM and the antecedent dry period was defined as the time since the previous rainfall event with a volume greater than 0.10 inches (0.25 cm).

Only four of the eight parameters were entered into the multiple regression equations developed with the forward regression program for the predictions of ΔDO or T. Table 13 lists the sequence of variables added in building the regression equation. The first variable entered was the CSO volume which accounts for the largest amount of variance in the dependent variable for a one variable model. This variable accounted for approximately 67 percent of the variance in the ΔDO data as indicated by the coefficient of determination, R^2. This result is reasonable since the change in DO due to sediment scouring should be highly related to the volume of CSO discharged.

The second variable entered into the equation was the natural log of the antecedent dry period. This two variable model accounted for about 76 percent of the variance in the dependent data. The rainfall volume was the

TABLE 13. RESULTS OF THE FORWARD (STEPWISE) REGRESSION ANALYSIS WITH ΔDO AS THE DEPENDENT VARIABLE

No. variables in equation	Independent variables entered	Intercept B_o	Variable coefficient B_1	Coefficient of determination R^2	Prob > F
1	CSO volume	7.62	26.22	0.673	0.001
2	CSO volume Log_e dry period	-3.49	27.56 2.30	0.761	0.001
3*	CSO volume Log_e dry period Rainfall volume	-0.90	76.95 2.77	0.810	0.001
4	CSO volume Log_e dry period Rainfall volume Dry period	-4.65	82.99 4.07 -39.15 0.01	0.820	0.001

*Selected model for prediction of ΔDO.

TABLE 14. RESULTS OF THE FORWARD (STEPWISE) REGRESSION ANALYSIS WITH T AS THE DEPENDENT VARIABLE

No. variables in equation	Independent variables entered	Intercept B_o	Variable coefficient B_1	Coefficient of determination R^2	Prob > F
1	Rainfall volume	15.58	23.33	0.287	0.022
2	Rainfall volume Dry period	7.89	27.68 0.02	0.425	0.016
3*	Rainfall volume Dry period CSO volume	5.90	43.57 -23.37 0.02	0.428	0.045
4	Rainfall volume Dry period CSO volume Log_e dry period	13.70	53.54 -36.70 0.03 -2.44	0.434	0.095

*Selected model for prediction of T.

third parameter added to the equation and the three variable model accounts for 81 percent of the variance in ΔDO. The four variable model accounted for 82 percent of the variance with the fourth variable being the antecedent dry period without the log conversion. No other variables met the 0.5 significance level for entry into the model. The values of the regression F-test (Probe > F column) indicate that the null hypothesis (that there is no correlation) can be rejected at the 99 percent confidence level for each of the models listed.

The three variable model was selected in order to predict ΔDO for two reasons:

1. The four variable model accounted for little additional variance in the dependent variable.

2. It did not seem practical to include the antecedent dry period and the natural log of the same data in the same predictive equation.

Prediction of the duration of the DO decline (T) was not as successful as for ΔDO. The duration is more difficult to predict due to changes in river flow and the influx of water from Lake Michigan. Table 14 lists the results of the regression analysis. In this case, the rainfall volume was the first variable entered into the model accounting for only 29 percent of the variance in the T data. The antecedent dry period was the second parameter added to the model. This two variable model accounted for 42 percent of the variance in T. The addition of the CSO volume and the natural log of the antecedent dry period explained little additional variance in the dependent variable. The four variable model was eliminated from consideration for predictive purposes because the null hypothesis could not be rejected at the 95 percent confidence level. The three variable model was the selected model because it did meet this criteria.

From the regression analysis, it was possible to predict the magnitude and the duration of the DO impact due to sediment scouring. The incorporation of this methodology into the receiving water quality model is discussed in Section 6.

CONTRIBUTION OF CSO

In order to fully evaluate the water quality impact of CSO, the pollutant contribution to the Milwaukee River from CSO was identified. In addition, the evaluation of pollutant loadings to the river also required consideration of the impact of CSO on the sediments, since both monitoring and modeling studies indicated that sediments are a major water quality influence in the Milwaukee River.

The pollutant loadings from CSO were first compared to the pollutant load entering the study area at the USGS gauging station. The CSO loadings were estimated with STORM while the upstream loadings were estimated from monthly sampling data and average monthly river flows from USGS (22). The 1977 water year (October, 1976 through September 1977) was evaluated since complete water quality data for the Estabrook Park site was available from USGS. The estimated annual loadings of selected pollutants is presented in Table 15.

TABLE 15. ESTIMATED ANNUAL POLLUTANT LOADINGS FOR THE 1977 WATER YEAR

Source	Suspended solids, lbx10^4	Carbonaceous BOD[1], lbx10^4	Total nitrogen, lbx10^4	Total phosphorus, lbx10^4
Upstream[2]	1420	314	89	7
CSO area	810	263	22	9

[1] Carbonaceous BOD is estimated as 1.5 times BOD_5 values for sanitary sources including CSO.

[2] Loadings represent values for the Milwaukee River upstream of the CSO area contributions. kg = 2.2xlb

Calculation of the annual pollutant loadings which contributed to the sediments was more difficult and various assumptions were required throughout the evaluation. An attempt was made to be consistent in all calculations and to provide a conservative or worst case estimate of the loadings. Therefore, it must be emphasized that the estimates are an approximation of the sediment loadings which are consistent with the other results of this study.

Suspended solids contributions to the sediments were calculated from results of 48-hour settleability tests on river water samples and composited CSO samples. The settleable percentage was assumed to represent the fraction of the suspended solids load that would contribute to the sediments in the study area of the Milwaukee River. The settling tests indicated that an average of 50 percent of the solids settled from ten river samples while an average of 98 percent of the solids settled from five CSO samples. Therefore, estimated suspended solids loadings to the sediment (Table 16) indicate that the upstream portion of the river and CSO contribute approximately equal proportions of suspended solids to the sediments.

TABLE 16. ESTIMATED LOADINGS TO THE SEDIMENTS IN THE MILWAUKEE RIVER STUDY AREA FOR THE 1977 WATER YEAR

Pollutant	Upstream	CSO area	*In situ* productivity
Suspended solids, lbx10^4	710	790	-
Carbonaceous BOD, lbx10^4	72	210	118-249

kg = 2.2xlb.

The loadings of oxygen demanding material were estimated in terms of carbonaceous BOD (CBOD). The relationships of soluble and insoluble BOD_5 and settling solids were used to estimate the BOD contributions to the sediments (Table 16). Total and soluble BOD_5 values were determined in river water and flow composited CSO samples. An average of 80 percent of the BOD in the CSO samples was associated with suspended material while about 25 percent of BOD in the river water was solids associated. The data and settling results established final CBOD loadings. This approach may overestimate the upstream contribution of BOD to the sediments since no data is available to establish whether the suspended BOD is associated with the settleable or non-settleable suspended solids.

Phytoplankton and macrophytic plants also contribute significant amounts of oxygen demanding materials to the sediments. On the basis of studies conducted by Bothwell (23) and field productivity measurements, the BOD contributed to the sediments from phytoplankton within the study area of the Milwaukee River has been estimated (24). It was assumed that all production reached the sediments, however, contributions of upstream sloughed plant material and phytoplankton were not added. The range of calculated BOD contribution from in situ productivity is also presented in Table 16.

These calculations have shown that roughly 40 to 50 percent of the annual loadings of oxygen demanding materials (in terms of CBOD) to the Milwaukee River are due to CSO. These data are consistent with the results of the chemical analyses of sediment samples, which indicate that the chemical composition of the sediments changed abruptly where the river enters the CSO area. However, since there are significant assumptions involved in the calculations, the values should be considered rough estimates with a wide confidence range. The numbers can be used to compare the loadings contributions from the different pollutant sources.

INSTREAM MONITORING RESULTS

The analysis of the data from the intensive surveys conducted during this project can be related to the findings of the sediment investigations for the DO results and to the dye studies for the fecal coliform results. Thus, low intensity rainfalls that cause overflows which do not result in widespread scour of the bottom materials will cause only a gradual decline in DO conditions in the lower reaches of the Milwaukee River. High intensity storms with long periods of dry weather prior to the event can cause the rapid loss of DO that is common to the lower reaches of the river. Further details of the individual monitoring events will be discussed in Section 7, Evaluation of CSO Impact.

The fecal coliform data obtained during the intensive monitoring surveys is not affected by the sediment investigations or the mechanisms explained in the preceding sections. However, the dye studies allow a better insight into the relation of coliform concentration and river flow. Of course, the number of coliforms discharged for each event is a function of the rainfall and prestorm history. But the length of time that the coliform standards are violated is a function of how fast the river flow is capable of flushing these organisms out of the lower portions of the river. The dye studies have indicated that the lake influence is minimal during the high flow conditions within the river. This accounts for the rapid decrease in coliform levels after the overflows and upstream contributions to the lower reaches of the river have ceased. Implementation of the mixing characteristics of the CSO outfalls into the coliform routines of the instream model could not be accomplished in this project. The instream sampling techniques near CSO outfalls was modified to insure that the entire cross section of the river was sampled and not just one location which could be influenced by a nearby outfall. The contribution of CSO to the coliform levels observed in the monitoring program is discussed in Section 6.

SECTION 6

MODELING

STORM MODEL

The Corp of Engineers Storage Treatment Overflow Runoff Model (STORM) was used in this study to simulate the existing sewer system (25). STORM is a "pipeless" model which does not route flow and quality through an actual sewer network but rather estimates flow and quality from a specified watershed by calculating total surface runoff and adding this flow to the watershed's total dry weather flow. Surface runoff and the associated pollutant washoff from the watershed are calculated by the model on an hourly basis, using a continuous hourly precipitation record, available surface pollutants and watershed characteristics (types of land uses within the watershed). A treatment rate, representing the flow diverted to the sewage treatment facilities, is applied to the total estimated flow (dry weather flow plus surface runoff) and any flow exceeding the treatment rate is considered overflow. An overflow event begins with the first hour in which overflow occurs and ends with the hour in which no overflow occurs. A rain storm lasting several hours could cause one or several overflow events, depending on the treatment rate, total rainfall for each hourly segment and watershed characteristics.

Each hour of overflow has an associated pollutant load, which depends on the surface pollutants available at the beginning of each hour and the hourly surface runoff rate. The pollutant loads simulated with the STORM model for this study were suspended solids, carbonaceous BOD (CBOD) and fecal coliform. CBOD was assumed to be approximately 1.5 times the BOD_5 for sanitary sources including CSO. The CBOD for separate storm sewer discharges was assumed to be slightly more than the BOD_5 for modeling purposes.

Rainfall

The results of the STORM simulations presented in this report were obtained by using continuous hourly precipitation data obtained from a tape supplied by the Southeastern Wisconsin Regional Planning Commission (SEWRPC). This tape contained 38 years of rainfall data for the period 1940 to 1977. The rain data was collected and recorded by the National Weather Service Office at Mitchell Field in Milwaukee.

Precipitation records from city raingages were used in lieu of the SEWRPC tape for the calibration process for simulations involving individual outfalls. The response of each outfall's drainage area to a particular storm event could best be defined using the city raingage located within each drainage area. A valid comparison between monitored data and model simulation results could then be made. Therefore, calibration was performed using the raingauge in the closest proximity to the outfall being evaluated.

Calibration

Before using STORM to simulate various sewer systems analyzed in this study, certain variables within the model had to be adjusted or calibrated so that the model output reflected characteristics which are specific to the Milwaukee area. The calibration procedure consisted of the following steps:

1. Collecting and tabulating monitoring data of CSO's in the Milwaukee area.
2. Applying STORM to the representative drainage area and rainfalls for which monitoring data exists.
3. Comparing output of the model with actual monitoring data.
4. Adjusting certain parameters in STORM until the output from the model comes close to actual measured values.

The monitoring data available for the calibration was from the following CSO sites:

27th Street on the Kinnickinnic River
Burleigh Street on the Milwaukee River
Edgewood Street on the Milwaukee River
Humboldt Avenue Detention Tank on the Milwaukee River.

These sites are shown in Figure 37.

Because STORM is a "pipeless" model and uses a single treatment rate to characterize a drainage area, a treatment rate for combined sewer overflow based on existing sewer system diversion structure capacities was impossible. The problem involves trying to characterize a sewer system which has dynamic flow dividers and diversion structures with a single treatment rate. A treatment rate equal to the peak dry weather flow was, therefore, used in simulating existing CSO conditions. This treatment rate was calculated using the formula for dry weather flow provided in the STORM manual (25). The calculation of dry weather flow is based on area size, population, land use types within a watershed, and infiltration. Population within each watershed modeled was obtained from Milwaukee census data and acreages used were derived by design engineers for the CSO system and from City of Milwaukee sewer maps. The land use types within a watershed were obtained from zoning maps. Five categories of land use types are commonly used and these include:

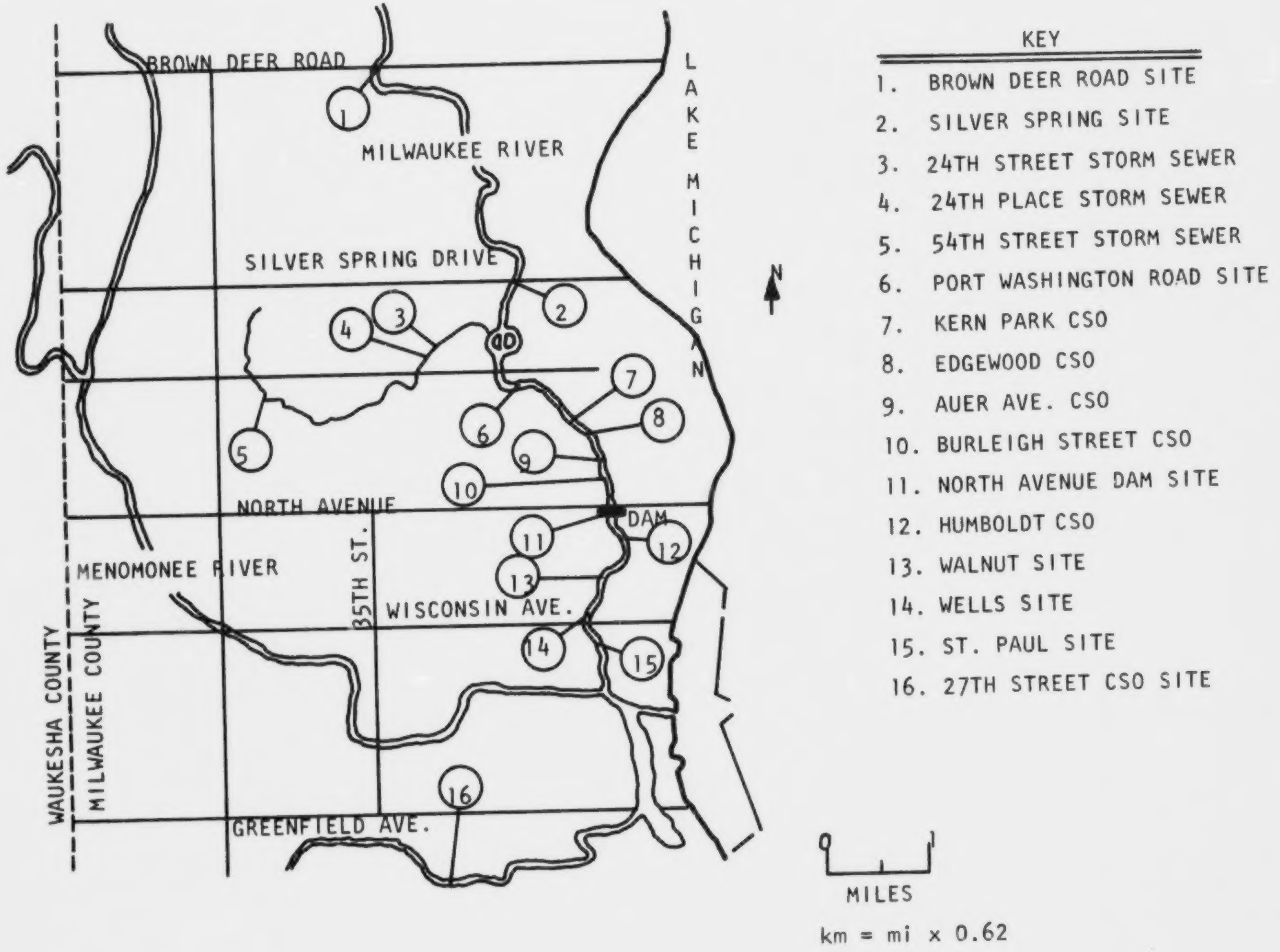

BROWN DEER ROAD
MILWAUKEE RIVER
SILVER SPRING DRIVE
NORTH AVENUE
MENOMONEE RIVER
35TH ST.
WISCONSIN AVE.
GREENFIELD AVE.
WAUKESHA COUNTY
MILWAUKEE COUNTY
LAKE MICHIGAN
DAM
N
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
KEY
1. BROWN DEER ROAD SITE
2. SILVER SPRING SITE
3. 24TH STREET STORM SEWER
4. 24TH PLACE STORM SEWER
5. 54TH STREET STORM SEWER
6. PORT WASHINGTON ROAD SITE
7. KERN PARK CSO
8. EDGEWOOD CSO
9. AUER AVE. CSO
10. BURLEIGH STREET CSO
11. NORTH AVENUE DAM SITE
12. HUMBOLDT CSO
13. WALNUT SITE
14. WELLS SITE
15. ST. PAUL SITE
16. 27TH STREET CSO SITE
0
1
MILES
km = mi x 0.62

Figure 37. CSO, storm sewer and river monitoring locations on the Milwaukee River, Lincoln Creek, and Kinnickinnic River.

1. Single family residential.
2. Multiple family residential.
3. Commercial.
4. Industrial.
5. Parks and open areas.

The infiltration rate provided in the STORM User's Manual (25) was used.

The calibration of STORM for the quantity of CSO for given rainfall events was not accomplished using monitored flows for several reasons. First, only "rough" flow measurements with stage boards were taken during monitoring. Secondly, modeling a single outfall drainage system with STORM is nearly impossible because STORM is a "pipeless" model. Nearly all of the CSO sewer networks are interconnected, making accurate predictions for a single outfall nearly impossible with STORM. Finally, it was not possible to monitor all of the outfalls with interconnected sewer networks. Therefore, STORM was calibrated for CSO quantity by comparing the predicted values for total overflow from STORM with values predicted by SWMM. This model was calibrated for the CSO drainage area as part of the Milwaukee Combined Sewer Overflow Pollution Abatement Project (11). Predictions from the two models for individual outfalls and groups of outfalls for several storms were compared. The runoff coefficients in STORM were adjusted so that the predicted overflow volumes were reasonable for several storm events.

Table 17 lists the predicted CSO volumes for individual CSO outfalls for a storm event on September 14, 1968. This storm was selected for comparison because it is a long duration event with variable rainfall intensities which produced 1.2 inches of rain in seven hours. STORM predictions of overflow volume were high for the Kern Park outfall and low for the Auer Avenue and Burleigh Street outfalls. However, the predictions seem reasonable.

TABLE 17. COMPARISON OF TOTAL FLOW PREDICTED BY THE SWMM AND STORM MODELS FOR THE SEPTEMBER 24, 1968 STORM.

Location	Total flow, $ft^3 x 10^6$ SWMM	Total flow, $ft^3 x 10^6$ STORM
Kern Park outfall	0.35	0.41
Auer Avenue outfall	0.87	0.60
Burleigh Street outfall	1.49	1.28

$m^3 = 0.028 x ft^3$

Storm sewer areas tributary to the Milwaukee River upstream of the CSO area contribution were also simulated with STORM. Figure 10 in Section 3 depicts the CSO and storm sewer areas simulated for input to Harper's water quality model.

When STORM was used to simulate flow and quality from the storm sewers a treatment rate was applied which was very close to zero. By using this treatment rate, essentially all surface runoff and its associated pollutant load reach the receiving waters. The same values for the variables used to control quantity of surface runoff in combined sewer simulations were used in storm sewer simulations. In addition to treatment rate, another hydrological difference in simulating storm sewers is the lack of dry weather flow. The dry weather flow block of STORM is used only in simulating combined sewer overflows.

STORM was set up using the dust and dirt method of pollutant accumulation which assumes that all pollutants are associated with dust and dirt accumulation in streets (lbs/100 ft of gutter length/day). A different dust and dirt accumulation rate is assigned to each land use category. Each dust and dirt accumulation rate has associated pollutant fractions, i.e., the fraction of suspended solids, BOD and fecal coliforms associated with each 100 lbs of dust and dirt.

In addition to adjusting the variables controlling the dust and dirt method of pollutant accumulation, the variables controlling the dry weather flow block of STORM were also used to calibrate the combined sewer overflow pollutant load predictions. The concentrations of pollutants in dry weather flow were obtained from the literature (22)(27). The model was calibrated to simulate dry weather flow which has a concentration of approximately 200 mg/l BOD, 250 mg/l suspended solids and $>10^6$/100 ml fecal coliforms. After these values for dry weather flow were obtained, the dust and dirt variables for surface runoff were then adjusted (calibrated) to obtain the desired overflow concentrations.

STORM was calibrated for quality using estimates of the composite quality concentrations for BOD_5 and suspended solids using the discrete field sample measurements and the relative discharge rates based on stage height. Fecal coliform counts were measured for discrete field samples and these provided a range to which the predicted composite fecal coliform counts could be compared.

The results of the quality calibration for CSO are presented in Tables 18 and 19. A wide variation in pollutant concentration exists for the monitored data. On a composite basis, monitored suspended solids ranged from 113 to 950 mg/l and BOD_5 ranged from 25 to 265 mg/l. These variations are due to differences in prestorm history of pollutant accumulation (number of dry days), rainfall volume and intensity, and the watershed characteristics of individual outfalls. It is not surprising that measured and modeled data do not always show a close fit. The model cannot be expected to predict accurately for each individual outfall on a per event basis using a single set of calibration variables. The model was calibrated to simulate overflow for the entire basin and should give reasonable predictions on an annual basis. On a yearly basis the model was calibrated for concentrations of approximately 400 mg/l suspended solids, 130 mg/l CBOD and $>10^6$/100 ml fecal coliform. These values fall within the range of the monitored data (Tables 18 and 19) and reported literature values (Table 20).

TABLE 18. STORM CALIBRATION RESULTS FOR SUSPENDED SOLIDS AND BOD COMPOSITE CONCENTRATIONS FOR CSO DISCHARGES

CSO Outfall	Date	SS, mg/l Measured	SS, mg/l Modeled	Measured (BOD_5), mg/l	Modeled (CBOD)[a], mg/l
Edgewood	7/6/77	210	70	55	89
	7/17/77	122	247	-	150
	8/4/77	113	70	25	100
Burleigh	7/23/75	145	424	50	158
	5/31/77	950	267	265	126
	6/17/77	524	109	170	56
	6/24/77	393	304	103	79
	6/28/77	418		26	
	7/6/77	405	101	79	81
	7/17/77	297	370	-	70
	8/4/77	281	238	60	83
Humboldt	9/12/72	228	304	63	124
	7/17/77	525	152	-	50
27th and KK	7/11/75	540	191	105	222
	7/23/75	265	308	41	235

[a] Assumed to be 1.5 x BOD_5 for sanitary sources including CSO.

TABLE 19. STORM CALIBRATION RESULTS FOR FECAL COLIFORMS FOR CSO DISCHARGES

Outfall	Date 1977	Fecal coliform, [counts/100 ml] x 10^3 Modeled (composite)	Fecal coliform, [counts/100 ml] x 10^3 Measured (discrete range)
Burleigh	5/31	6600	18-650
	6/17	1800	75-12,000
	6/24	2900	3-TNTC[a]
	6/28	2100	15-600

[a] Too numerous to count.

TABLE 20. LITERATURE VALUES FOR COMBINED WASTEWATER AND SEPARATED STORM FLOW DISCRETE QUALITY

Location	BOD_5, mg/l		Suspended solids, mg/l		Fecal coliform, [counts/100 ml] x 10^3	
	Range	Mean	Range	Mean	Range	Mean
	Combined wastewater					
Washington, DC (28)	10-470	71	35-2,000	622	240-5,040	2,400
Detroit (29)		50		250		
Detroit (29)		153		274		
Detroit (30)			23-1398	150	200-17,000[a]	
	Separated storm flow					
Washington, DC (28)	3-90	19	130-11,280	1,697	40-1,300	310
Detroit (29)		147		147		
Cincinnati (29)		19		210		
Ann Arbor (29)		28		2,080		
Detroit (30)			900-2,062		8-1,115[a]	

[a] Median values from several samples taken during a particular flow event.

The quality of storm sewer flow predicted by STORM is solely a function of surface runoff quality which is determined by the variables controlling the dust and dirt method of calculating pollutant accumulation. These variables were adjusted during calibration. The model was calibrated for storm sewer concentrations of approximately 100 mg/l suspended solids, 30 mg/l CBOD and >10^3/100 ml fecal coliform. The CBOD and fecal coliform values are consistent with the monitored data (Table 21) and literature values (Table 20). However, the calibrated suspended solids concentrations are higher than the measured values (Table 21). This is because all the monitored storm sewers are in single family residential areas, which have lower suspended solids concentrations than commercial or industrial locations. This is supported by the fact that storm sewer suspended solids concentrations reported in the literature were considerably higher. Therefore, a suspended solids concentration of 100 mg/l was used to characterize the area being modeled by STORM.

TABLE 21. CONCENTRATIONS FOR MONITORED STORM SEWER SAMPLES

Location	Date 1977	Suspended solids, mg/l (composite)	BOD_5, mg/l (composite)	Fecal coliforms, [counts/100 ml]x10^3 (discrete range)
54th Street	6/17	34	11	1-70
	6/28	11	14	3-18
	7/7	31	14	
	7/17	34	-	
24th Street	7/7	19	14	
	7/17	60	-	
24th Place	7/7	18	15	
	7/17	45	-	

RECEIVING WATER QUALITY MODEL

General Assumptions

The water quality model modified for this project is based on a model used in previous studies on the Milwaukee River (11). Similar to its predecessor, the modified model is a series of equations applied to the section of the Milwaukee River extending from Brown Deer Road to the confluence of the Milwaukee River with the Menomonee River as shown in Figure 37. The modeled section of the river is broken into several segments or reaches. The characteristics of these reaches are assumed to be constant within each reach for a certain time period. Characteristics may change from reach to reach or within the same reach after a certain period of time has passed. The characteristics include physical and biological variables such as sediment oxygen demand, reaeration and deoxygenation rate coefficients, mean velocity and average water depth.

The water quality model is a one-dimensional, time-varying model. Complete vertical and lateral mixing of all water quality parameters is assumed. Thus, variations of dissolved oxygen or temperature, with depth, which have been observed in the Milwaukee River, are not simulated. Similarly, variations in water quality parameters across the river, perpendicular to the direction of flow, are also not modeled. The model calculates changes in water quality parameters along the length of the river, or longitudinally, as a function of time.

Hydraulic Equations

Hydraulic data needed by the water quality model must be developed separately and input to the model. This data includes the hydraulic characteristics of the river reaches;

Mean width.
Depth.
Discharge.

When modeling unsteady flow conditions, these characteristics may change during the simulation. This requires a new set of hydraulic data cards for each flow condition. The hydraulic characteristics are held constant within each reach, however, until changed by a new set of hydraulic data cards.

Routines are programmed in the water quality model for linear interpolation of hydraulic data. This interpolation is necessary to minimize numerical errors caused by immediate changes of hydraulic conditions at the boundaries of river reaches and when hydraulic conditions change for a particular reach. If not corrected, numerical errors could result in simulations in which large amounts of pollutants are either gained or lost when, in fact, no change is desired.

Each river reach is divided into several units called computational elements for solving the equations of the water quality model. Three hydraulic factors may vary within the river reaches and are determined for each computation element. These factors are inflow from Lake Michigan, overflow volumes from combined sewers and inflow volumes from surface runoff.

Inflow from Lake Michigan is considered a uniformly increasing discharge beginning at some point downstream from the North Avenue Dam to a specified value at the confluence of the Milwaukee and Menomonee Rivers (Figure 38). The basis for the development of this relationship was trends in salt water intrusion in estuaries. Also, monitoring data on specific conductance and total dissolved solids during dry weather collected for a previous project (11) are generally in agreement with this relationship. Evaluation of the rate of inflow from Lake Michigan is discussed later in this section of the report.

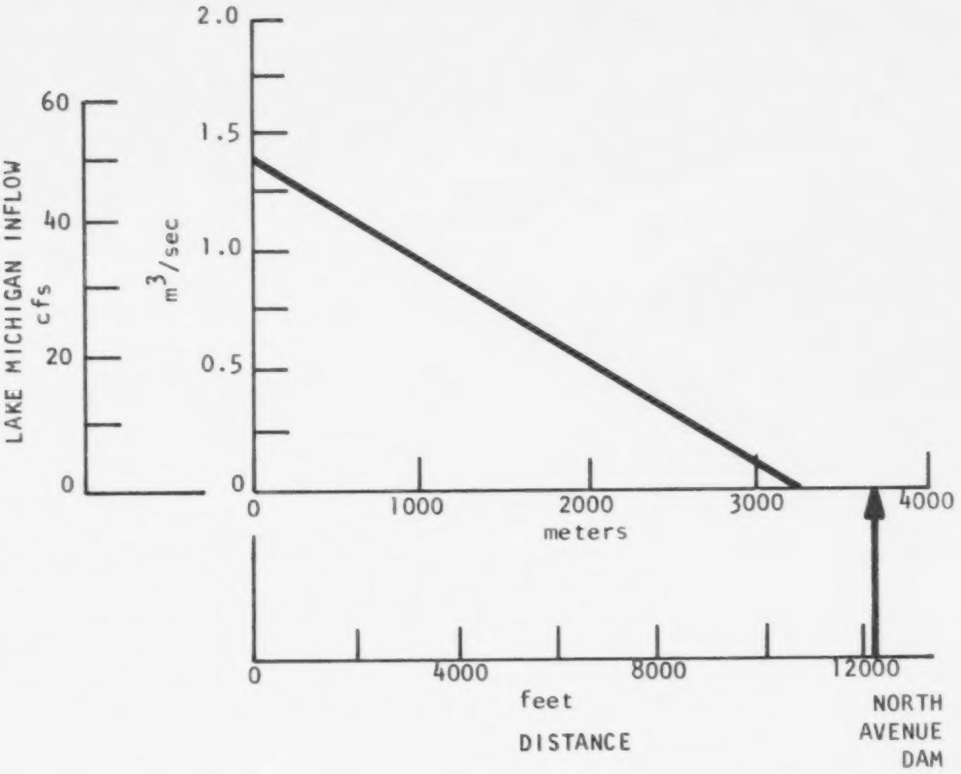


Figure 38. Illustration of assumptions utilized in Harper's water quality model for simulation of the Lake Michigan inflow.

Overflow volumes from combined sewers and inflow volumes from surface runoff are calculated from data supplied by either SWMM or STORM. The water quality model is capable of receiving inflow data from several storms and is presently programmed for twenty storms. This restriction may easily be increased by increasing storage in the appropriate statements in the program. Another restriction is that the number of inflows (the sum of combined sewer outfalls and storm runoff discharge points) must remain constant for the simulation period and is limited to 80 in number.

Water Quality Equations

The water quality model is a series of equations which are linked to calculate changes in several water quality parameters. These water quality parameters are any conservative pollutant (i.e., total dissolved solids), temperature, phytoplankton, biochemical oxygen demand, dissolved oxygen and fecal coliform bacteria. Equations for growth, respiration and sloughing of benthic algae are also included in the model.

Basic Transport Equation--
The basic equation used to simulate transport of all materials in the Milwaukee River is referred to as the convective-dispersion equation:

$$\frac{\partial C}{\partial t} + V\frac{\partial C}{\partial x} = D_L \frac{\partial^2 C}{\partial x^2} \pm S(x,t) \ldots\ldots 1$$

Where:

C = Water quality parameter concentration.
t = Time.
V = Stream velocity.
D_L = Coefficient of longitudinal dispersion.
x = Longitudinal distance.
S = Source-sink term for the water quality parameter.

This equation describes the rate of change of a water quality parameter based on the assumptions of complete lateral and vertical mixing as previously discussed. For conservative parameters, such as total dissolved solids, the source-sink term equals zero at all times and locations. The source-sink term varies for each parameter according to the following discussion.

Phytoplankton and Benthic Algae Source-Sink Equations--
The growth and respiration rates of phytoplankton (floating and suspended algae) are assumed to apply for the growth and respiration of benthic (attached to the river bottom) algae. Adjustments are made for the light available for growth of benthic algae and for sloughing of benthic algae from the stream bottom. The following discussion, however, basically applies to both phytoplankton and benthic algae, even though only phytoplankton are mentioned.

The equation used to estimate the changes in phytoplankton concentrations is a first order linear differential relationship:

$$S_p\ (x,t) = \frac{dP}{dt} = (U-R-Set)P \ldots\ldots 2$$

Where:

P = Phytoplankton concentration, mg dry weight algae/l.
U = Phytoplankton growth rate, sec^{-1}.
Set = Phytoplankton settling rate, sec^{-1}.
R = Phytoplankton respiration rate, sec^{-1}.
$S_p\ (x,t)$ = Net phytoplankton growth, mg dry weight algae/l-sec.

Mathematical models have usually treated phytoplankton growth rate as a function of several nutrients, light and temperature. The nutrients have included nitrate, ammonia, phosphorus and carbon. These models have equations which reduce the growth rate of phytoplankton from some maximum rate to a lower rate depending upon the modeled concentrations of nutrients and relative optimum levels required by algae. Light available for growth is normally treated in a manner similar to nutrients.

Existing nutrient concentrations in the Milwaukee River are normally higher than concentrations usually considered limiting to algal growth. Consequently, phytoplankton growth is assumed to be limited by available light and the growth rate equation may be expressed as:

$$U = U_{max}\frac{(I_{ave})}{(K_I + I_{ave})} \tag{3}$$

Where:

U_{max} = Maximum growth rate, a function of temperature, sec^{-1}.
K_I = Half saturation constant for light, Langley sec^{-1}.
I_{ave} = Average light intensity through the water column, Langley sec^{-1}.

Average light intensity through the water column is used in the growth rate equation because phytoplankton are assumed to be uniformly distributed through the water column consistent with the mixing assumptions. The average light intensity (I_{ave}) can be shown to be related to the incident light intensity (I_o) by the relationship:

$$I_{ave} = \frac{I_o}{kH}\,(1-e^{-kH}) \tag{4}$$

Where:

I_o = Incident solar radiation, Langleys sec^{-1}.
H = Mean stream depth, ft.
$k = k_b + (K_{phyto})\ (P)$ = extinction coefficient, ft^{-1}.
k_b = Base extinction coefficient, ft^{-1}.
K_{phyto} = Coefficient for variable extinction coefficient (mg/l-ft)$^{-1}$.

The variable extinction coefficient is intended to simulate the self-shading of algae. Through this mechanism, high phytoplankton concentrations will absorb most of the light in the water column thereby reducing light available for growth to levels which limit phytoplankton growth.

Both maximum growth rate, U_{max} and respiration rate, R, are assumed to be functions of temperature. Linear relationships are used for both variables based on data from literature (31) which are illustrated in Figure 39.

The phytoplankton settling rate is a variable which is estimated for each river reach. Establishment of this rate is part of the model calibration procedure. It is noted that the net growth (U-R) P is used in calculation of dissolved oxygen concentrations. In the sequence of computations, phytoplankton growth and respiration occur prior to settling. This sequence of calculations was found to be necessary during model calibration to obtain

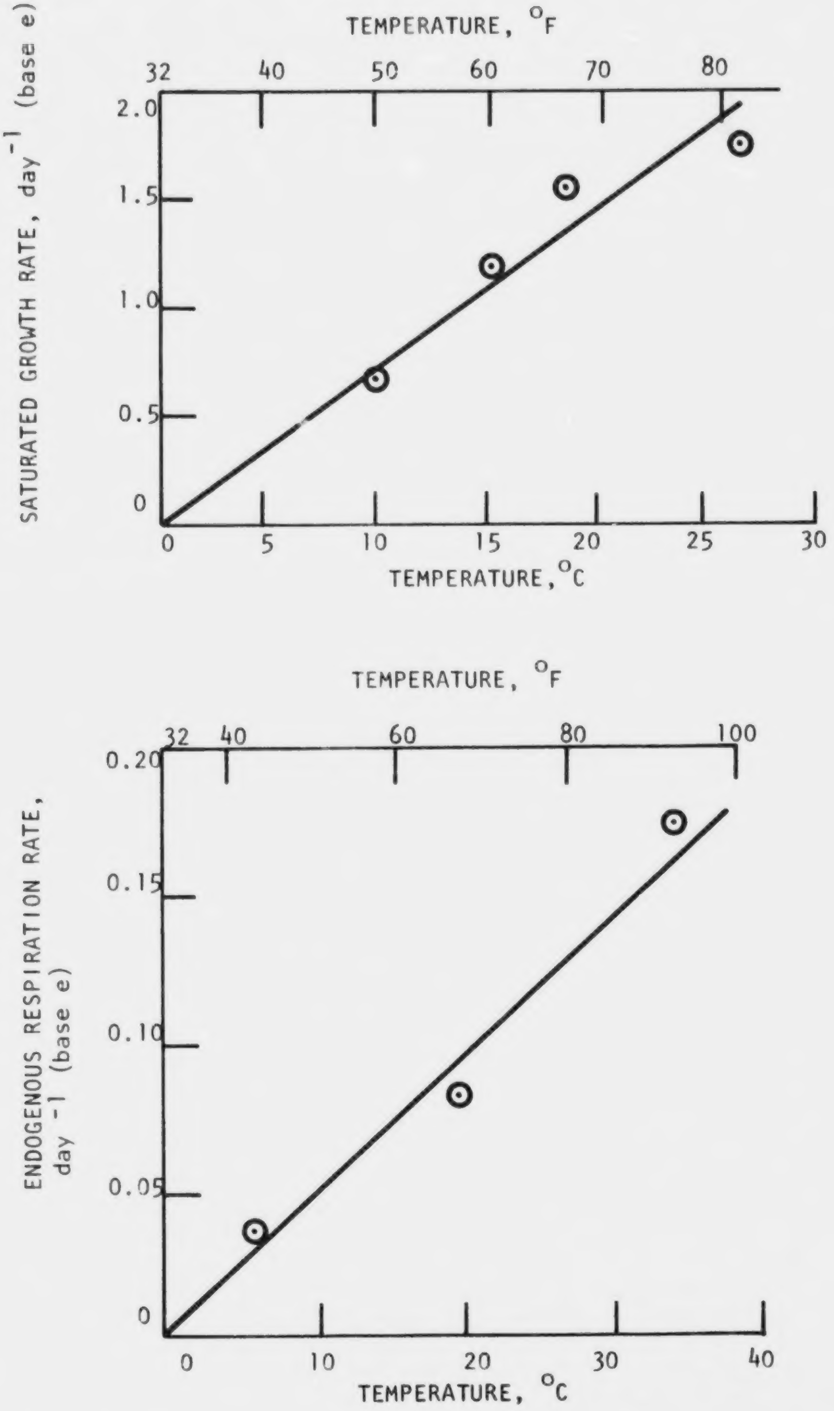


Figure 39. Influence of temperature on the maximum growth rate of phytoplankton (U_{max}) and phytoplankton endogenous respiration rate (31).

observed diurnal ranges in dissolved oxygen while maintaining acceptable levels of phytoplankton.

The kinetics of benthic algal growth are similar to phytoplankton growth kinetics. Major differences are:

1. Benthic algae losses are modeled by sloughing off rather than by settling.

2. Light reaching the channel bottom is used in the growth rate expression rather than average light intensity through the water column.

A change in units from the phytoplankton growth equation (equation 2) is necessary as benthic algae are expressed on a dry weight per unit area basis rather than a dry weight per unit volume basis. That is:

$$S_b\ (x,t) = \frac{dB}{dt} = (U-R)\ B \quad \ldots\ldots 5$$

Where:

B = Benthic algae concentration, mg/m^2.
$S_b\ (x,t)$ = Net benthic algae growth, mg/m^2-sec.

Sloughing of benthic algae is simulated by restricting the maximum allowable density of benthic algae. Thus, net growth may occur when the density is at the maximum level, but the density will not increase. This maximum level is presently programmed as a single value specified by the user for the entire river being modeled.

Dissolved Oxygen Source-Sink Equations--
The sources and sinks of oxygen considered to be most significant in the mathematical model are:

1. Reaeration.
2. Oxygen consumption by bottom deposits and suspended and dissolved material.
3. Algal production and respiration of oxygen.

Reaeration is assumed to be a first order process where the rate of transfer across the air-water interface is proportional to the dissolved oxygen deficit. The dissolved oxygen deficit is the difference between the actual concentration and the dissolved oxygen concentration for saturation conditions. If super-saturation conditions occur, the transfer of oxygen is from the water column to the atmosphere. If undersaturation conditions occur, or if a dissolved oxygen deficit exists, oxygen is transferred from the atmosphere to the water column. This process may be expressed by:

$$\frac{dC}{dt} = k_2\ (C_s - C) \quad \ldots\ldots 6$$

Where:

C = Dissolved oxygen concentration, mg/l.
k_2 = Reaeration coefficient (a function of temperature) sec^{-1}.
C_S = Saturation dissolved oxygen (a function of temperature), mg/l.

$$C_S = 14.652 - 0.41022T + 0.007991T^2 - 0.000077774T^3 \tag{7}$$

Where:

T = Stream temperature, °C.

The reaeration coefficient may be estimated from several published experimental equations. The O'Connor-Dobbins equation is used in this model (32). Isotropic turbulence conditions or conditions when the Chezy coefficient is greater than 20 must be assumed. This equation has been found to be generally adequate for most river applications. Problems in using the equation have been experienced for rivers having average depths less than one foot. In these cases, the applicability of the equation depends on the time step used to solve the equations of the model.

The reaeration coefficient equation for 20°C is:

$$k_2 = \frac{(D_m \overline{V})^{1/2}}{H^{3/2}} \tag{8}$$

Where:

D_m = Molecular diffusivity of oxygen at 20°C, ft^2/sec.
$\overline{V}$ = Mean stream velocity, fps.

The reaeration rate coefficient is corrected using a Van't Hoff-Arrhenius type expression:

$$k_2(T) = k_{2(20)}\theta_1^{(T-20)} \tag{9}$$

Where:

$k_2(T)$ and $k_2(20)$ = Reaeration coefficients at T°C and 20°C, respectively.
θ_1 = A constant ranging from 1.01 to 1.047.
T = Stream temperature, °C.

The sediment oxygen demand (or benthal oxygen demand) is estimated differently for dry weather conditions and wet weather conditions. This difference is due to the immediately higher oxygen demand observed to occur following combined sewer overflows. This higher demand cannot be solely attributed to biologically controlled reactions which was explained in Section 4 and 5.

Sediment oxygen demand during dry weather conditions is calculated by an equation which sets the benthic demand constant until the dissolved oxygen in the overlying water limits oxygen uptake (21):

$$\frac{dC}{dt} = \frac{k_3 C^{0.3} \, \Theta_2^{(T-20)}}{H} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 10$$

Where:

k_3 = Consumption rate coefficient, mg/l-sec.
C = Dissolved oxygen concentration of overlying water, mg/l.
Θ_1 = Constant for temperature correction ranging from 1.067 to 1.078.

The coefficient k_3 is a function of the type, depth and age of deposited material and of the types of organisms decomposing the organic fraction. A Van't Hoff-Arrhenius equation is used to adjust the oxygen consumption as the temperature changes.

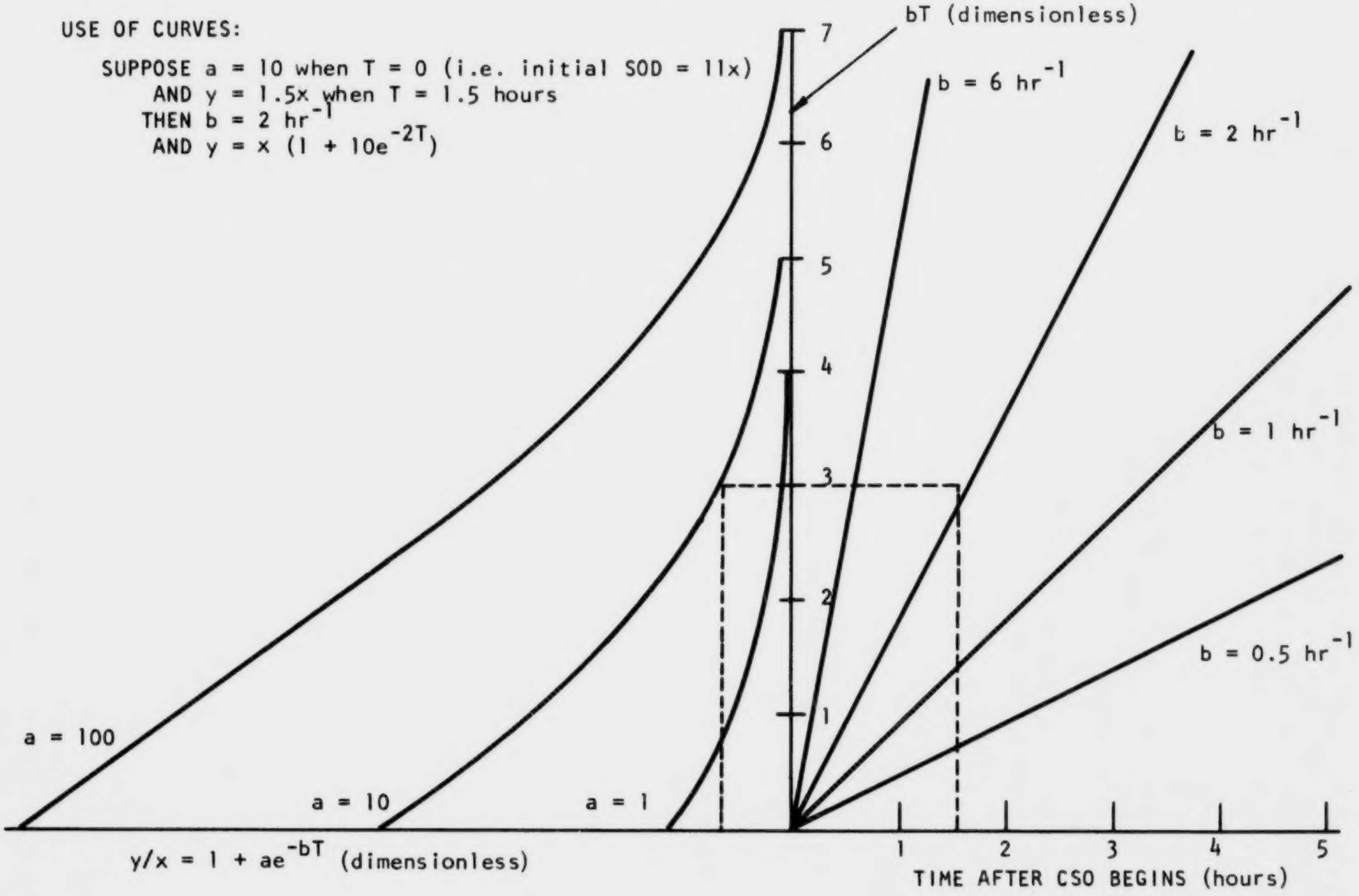
The wet weather sediment oxygen demand is modeled as a time-varying demand. This demand is initially calculated as a multiple of the dry weather demand to approximate the immediately high demand required at the time a combined sewer overflow event begins. The high demand is reduced at an exponential rate to the dry weather sediment oxygen demand. The magnitude of the increase in sediment oxygen demand and rate of decrease are specified by the model user for each CSO event. The equation describing the time-varying sediment oxygen demand is:

$$\text{SOD wet weather} = (\text{SOD dry weather})\,(1 + ae^{-bt}) \ldots\ldots\ldots\ldots\ldots\ldots 11$$

Where:

a and b are constants which may be varied for each CSO event and t is the simulation time after the CSO event has begun.

The constants a and b are intended to reflect the range in wet weather SOD rates. For example, a long dry weather period preceeding a CSO event would probably cause a larger increase in wet weather SOD than a shorter dry period due to more time for production of gases and reduced chemicals in the sediments under anaerobic conditions. Similarly, a more severe CSO event would probably cause a larger wet weather SOD due to more disturbance of the sediments. Thus, for these instances, the value of the "a" constant would be larger than for shorter dry weather periods or less severe CSO events. The "b" constant, which regulates the rate at which the wet weather SOD is reduced to the dry weather SOD, may be similarly varient according to the nature of the CSO event. The curves shown in Figure 40 illustrate the characteristics of the time varying SOD equation.



USE OF CURVES:

SUPPOSE $a = 10$ when $T = 0$ (i.e. initial SOD = 11x)
AND $y = 1.5x$ when $T = 1.5$ hours
THEN $b = 2\ hr^{-1}$
AND $y = x\ (1 + 10e^{-2T})$

Figure 40. Characteristics of the time varying sediment oxygen demand

The water quality model also has the flexibility to restrict the operation of the wet weather SOD equation to certain reaches. A river location may be specified above which the wet weather SOD equation should not apply. Moreover, the length of time that the wet weather SOD equation applies may be specified.

These factors in addition to the constants a and b are input to the model for each combined sewer overflow event. Analysis of continuously monitored dissolved oxygen conditions yields estimates of these factors for initial model calibration simulations.

Oxygen consumption by suspended and dissolved material is assumed to be the typical first order process:

$$\frac{dC}{dt} = -k_1 L \Theta_3^{(T-20)} \qquad 12$$

Where:

k_1 = Deoxygenation constant, day^{-1}.
L = Ultimate biochemical oxygen demand, mg/l.
Θ_3 = Constant for temperature correction approximately equal to 1.047.

The uptake and release of dissolved oxygen by phytoplankton and benthic algae are assumed to be proportional to the net growth of phytoplankton and benthic algae:

$$\frac{dC}{dt} = U_{Oz} (U-R) \left(P+\frac{B}{H}\right) \qquad 13$$

Where:

C = Oxygen concentration, mg/l.
U_{Oz} = Oxygen uptake rate coefficient, mg/mg.
U, R, P, B and H are as previously defined.

Adding all of the terms together yields the composite dissolved oxygen source-sink equation:

$$\frac{dC}{dt} = k_{2(20)}(C_s - C)\,\Theta_1^{(T-20)} + \frac{k_3 C^{0.3}}{H}\,\Theta_2^{(T-20)}$$

$$-k_1 L \Theta_3^{(T-20)} - U_{Oz}(U-R)\left(P+\frac{B}{H}\right) \qquad 14$$

Temperature Source-Sink Equations--
In the preceeding sections, several rate coefficients have been given or developed as functions of temperature, including the maximum algal growth rate, algal respiration rate, saturation dissolved oxygen concentration and the Van't Hoff-Arrhenius type temperature corrections in the oxygen

source-sink equation. Because of these interactions, water temperature is simultaneously modeled with the other water quality parameters.

One of the simplest methods available for temperature modeling is the heat balance approach published by Raphael (33). This procedure involves estimation of the net heat transfer across the water surface to calculate the change in temperature of the waterbody. The equation is:

$$S_T = \frac{dT}{dt} = \frac{Q_T}{C_p \gamma H} \quad \text{.....15}$$

Where:

Q_T = Net heat transfer across the water surface, Btu $ft^{-2} sec^{-1}$.
C_p = Specific heat of water, Btu $lb^{-1} {}^oF^{-1}$.
γ = Unit weight of water, 62.4 lbs ft^{-3}.
S_T = Rate of change in temperature, ^oF sec^{-1}.

The net heat transfer consists of several components, each of which is estimated independently. The individual components are incident solar radiation (Q_I), conductive heat transfer (Q_H), effective back radiation (Q_B), and evaporative heat transfer (Q_E). Incident solar radiation may be calculated theoretically using the solar altitude, the latitude of the site and the cloud cover. Direct measurement of incident solar radiation is also possible.

Evaporative heat transfer equations have been developed from Lake Hefner evaporation data. Wind speed and vapor pressure differences are the major variables used in the equation:

$$Q_E = K_E U(e_w - e_a) \quad \text{.....16}$$

Where:

K_E = Experimental constant.
U = Wind speed, knots.
e_w = Vapor pressure of water in saturated air at the temperature of the water surface, inches of mercury.
e_a = Ambient vapor pressure, inches of mercury.

The conductive heat transfer is estimated using the Bowen ratio of conducted heat to energy utilized by evaporation, which leads to:

$$Q_H = K_H UP (T - t_a) \quad \text{.....17}$$

Where:

K_H = Experimental constant.
P = Atmospheric pressure, inches of mercury.
t_a = Air temperature, ^oF.

Effective back radiation has been defined as the difference between the long wave radiation leaving a body of water and the long wave radiation from the atmosphere being absorbed by the body of water. Radiation from both of these bodies can be calculated using the Stefan-Boltzmann radiation law. Radiation from the water surface is a function of the temperature of the water body and the emissivity of the water surface. Atmospheric radiation is dependent upon the air temperature and the moisture of the air. Energy transfer by both mechanisms has been combined to yield the effective back radiation as:

$$Q_B = 0.97\sigma\,(T_w^{\,4} - \beta T_a^{\,4}) \ldots\ldots 18$$

Where:

σ = Stefan-Boltzmann constant, 1.74×10^{-9} Btu $ft^{-2} hr^{-1} {}^{o}R^{-4}$.
T_w = Absolute temperature of water body, ^{o}R.
T_a = Absolute temperature of air, ^{o}R.
β = Atmospheric radiation factor, a function of cloud cover and ambient vapor pressure.

A considerable amount of weather data is required to calculate the heat budget components. This information is routinely monitored and reported by the National Oceanographic and Atmospheric Administration at numerous sites in the United States. Weather data is reported at three hour intervals for Mitchell Field in Milwaukee and includes all of the necessary data for calculation of heat budget components.

Other sources and sinks of heat may exist in the heat balance. Heat advected into the stream by tributaries or wastewater inputs is treated by a mass-balance type of equation in the model. Other possible sources and sinks of heat such as heat loss by conduction to the ground and internal heat generation by chemical and biological reactions and friction losses are assumed to be negligible.

Fecal Coliform Bacteria Source--

The rate of change in fecal coliform counts is assumed to follow first-order kinetics:

$$\frac{d(Col)}{dt} = (k)(Col) \ldots\ldots 19$$

Where:

Col = Fecal coliform bacteria, counts/100 ml.
k = Net growth rate coefficient, sec^{-1}.

$$k = U_{max}\frac{L}{K_{cs} + L} - k_c \ldots\ldots 20$$

Where:

U_{max} = Maximum fecal coliform growth rate, sec^{-1}.
L = Biochemical oxygen demand, mg/l.
K_{cs} = Half-saturation constant, mg/l.
k_c = Die-off rate coefficient, sec^{-1}.

Example of Temperature and Dissolved Oxygen Equations

The temperature and dissolved oxygen equations are sufficient to model extremely complex aquatic systems. Some understanding of the response of the equations may be gained by application of the equations to a simplified river system. The results of such an application are illustrated in Figure 41.

Two simplified river systems are illustrated in Figure 41. The first consists of a stream which receives a constant discharge of oxygen demanding materials. The second system is a stream which supports algae; phytoplankton or benthic algae. In both instances, stream temperature varies diurnally with maximum temperatures occurring during afternoons and minimum temperatures occurring during earlier morning hours (Figures 41-a and 41-d).

For the system receiving the constant discharge, the impact of the discharge can be evaluated by examination of dissolved oxygen conditions upstream and downstream from the discharge. At the upstream station (Figure 41-b), dissolved oxygen concentrations are at saturation values, calculated according to equations 6 and 7. The dissolved oxygen curve is a mirror image of the temperature curve at the station. This is because cooler water has a greater capacity for dissolved oxygen than warmer water. Therefore, minimum dissolved oxygen concentrations occur at maximum temperatures and maximum dissolved oxygen concentrations occur at minimum temperatures. The mean dissolved oxygen (illustrated by the dashed line in Figure 41-b)Is the saturation concentration of the mean temperature (illustrated by dashed line in Figure 41-a). The diurnal range in dissolved oxygen (Δ_1) is caused solely by temperature variations.

Downstream from the location where the discharge enters the river (Figure 41-c) the entire dissolved oxygen curve has been moved downward by an average amount (Δ_2). This decrease is caused by the exertion of oxygen demanding materials. These materials may include dissolved or suspended biochemical oxygen demand from material discharged or released to the river (equation 12), or may include a constant demand from the sediments such as during dry weather (equation 10).

Phytoplankton and benthic algae will cause significant diurnal variations of dissolved oxygen directly in phase with water temperature variations (compare Figures 41-d and 41-f). This similarity is due to the importance of incident solar radiation in the heat budget equation (equation 15) and in the algal growth rate equation (equation 3).

The impact of wet weather discharges may also be discussed in terms of these examples. A large discharge of oxygen demanding materials to the

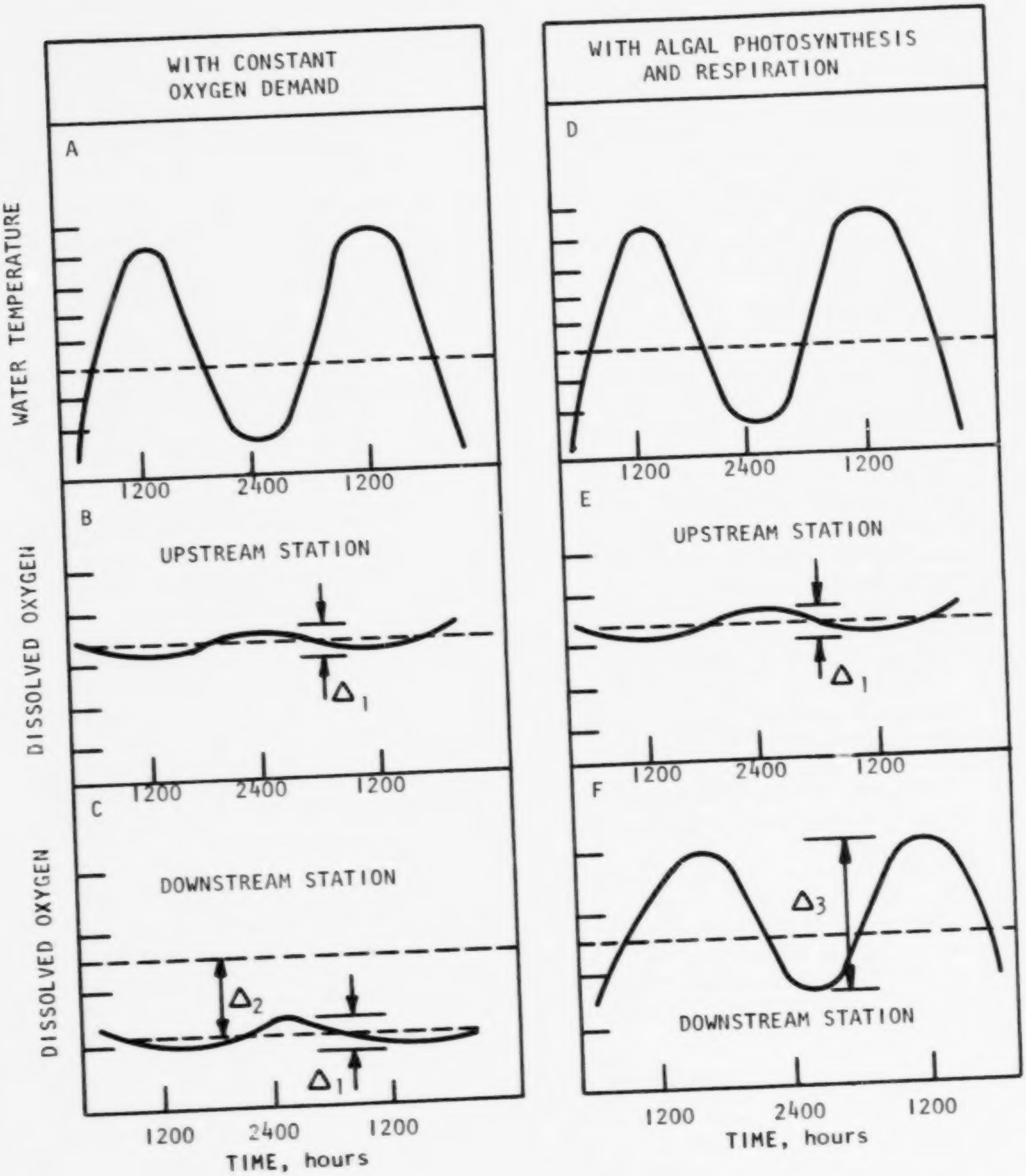


Figure 41. Example of temperature and dissolved oxygen simulations with Harper's water quality model.

river would cause a temporary effect similar to constant oxygen demand. The dissolved oxygen concentrations would be depressed below the concentrations shown for the downstream stations in both examples. The depression would occur for a short period of time and conditions would recover to the dry weather conditions. The large discharge of oxygen demanding materials may occur from combined sewer overflows or disturbance of sediments. The large demand may be caused by biological and/or chemical factors. Coefficients in equation 11 are adjusted as part of model calibration to simulate the observed conditions.

Understanding these relationships provides the basis for model calibration and verification. Rate coefficients in the equations may be initially estimated based on previous model applications. The water quality model can be more easily calibrated and verified, however, if the nature and importance of the various components of the equations are understood.

Receiving Model Set Up

Harper's water quality model was set up to simulate a 14 mi (22.6 km) portion of the Milwaukee River extending from Brown Deer Road to the confluence with the Menomonee River. For this study, calibration of the model was for sites downstream from Port Washington Road. The area upstream of the site was included in the set up in order to accurately simulate loadings from storm sewer areas tributary to the Milwaukee River and Lincoln Creek.

The river was segmented into 100 units, each being approximately 740 ft (225 m) in length. Input requirements for describing the river include the following:

1. Distance or length of river simulated.
2. Flow rate.
3. Mean width of segment.
4. Mean depth of segment.

The mean width and depth did not have to be specified for each of the 100 segments. Rather, these dimensions are specified at the upstream boundary and several locations downstream. Dimensions of the segments in between these locations are calculated by the computer program using linear interpolation. River dimensions in the Milwaukee River were specified at eight locations downstream of Port Washington Road and three locations upstream. Less detail was required upstream since this area was mainly utilized to input storm sewer loads. Specified dimensions in the model can be changed during the simulation as the flow rate changes. This provides the flexibility for accurate simulations of velocity in the river.

Other model input requirements for a given simulation include the following:

1. Upstream quality concentrations.
    Conservative parameter - usually total dissolved solids.
    Temperature.

Phytoplankton.
Biochemical oxygen demand.
Dissolved oxygen.
Fecal coliforms.
Benthic algae.

2. Initial quality conditions - several locations.
    Parameters same as above.

3. Lake Michigan influx flow and quality.
    Parameters same as above.

4. Rate coefficients.
    Base extinction coefficient for light.
    Sediment oxygen coefficient.
    BOD resuspension coefficient.
    Coliform decay.
    Phytoplankton settling coefficient.

5. Weather data.
    Solar radiation.
    Wind speed.
    Atmospheric pressure.
    Cloud cover.
    Dew point temperature.
    Air temperature.

6. Phytoplankton parameters.
    Self shading factor.
    Half saturation constants.
    Uptake rate coefficients.

7. Combined sewer and storm sewer loadings.
    Input time.
    Flow rate.
    Conservative - usually suspended solids.
    Temperature.
    Biochemical oxygen demand.
    Fecal coliforms.

8. CSO scour coefficients.
    Distance where coefficients apply.
    Magnitude coefficient.
    Decline coefficient.
    Decline time.

9. Size of time step.

The upstream quality is specified at time intervals throughout the simulation. This is also true of the weather input data. The CSO loadings are specified for the appropriate input time. Restrictions on the input time are:

1. That there must be an upstream quality specified at this time.
2. The input time must be a multiple of the time step.

The CSO scour coefficients are specified for each CSO loading. All other inputs remain constant throughout the simulation. Sensitivity analyses on the size of the time step were conducted to determine how large the time step could be without causing large computational errors. Figure 42 presents the results of this analysis for a one and four hour time step. These results indicate that computational errors do not get extremely large even when a time step of four hours is utilized. A three hour time step was selected for modeling purposes. This allows suitable flexibility for inputting CSO loads and for the length of simulations.

Receiving Model Calibration/Verification

Calibration/verification of Harper's model involved several steps:

1. Calibration of the water quality model for dry weather/steady flow conditions.

2. Development of a time varying SOD rate for simulation of sediment scouring and the subsequent release of oxygen demanding materials.

3. Development of a predictive procedure for the determination of time varying SOD coefficients based upon rainfall/runoff characteristics.

4. Calibration of the water quality model for wet weather/unsteady flow conditions.

5. Verification of the water quality model for wet weather conditions.

The development of the time varying SOD equations has been discussed previously in the discussion of Harper's water quality model. The development of a predictive methodology for determination of the coefficients of the time varying SOD equation was discussed in Section 5. The remaining topics will be discussed in the following paragraphs.

Dry Weather Calibration--

Harper's model was originally calibrated for dry weather as part of the Milwaukee Combined Sewer Overflow Abatement Project (11) utilizing data from an intensive dry weather survey in June, 1975. Data collected during dry weather in September, 1976 (Survey 1) for this project has been utilized to verify the initial calibration of the model, as well as determine an acceptable range for certain parameters for wet weather calibration. The September, 1976 survey is for that portion of the river in the CSO area (downstream of Capitol Drive).

Figures 43 and 44 and Appendix Figures A-9 and A-10 illustrate the verification results for dissolved oxygen at sites in the CSO area portion of the Milwaukee River for Survey 1. The range in DO shown for the observed

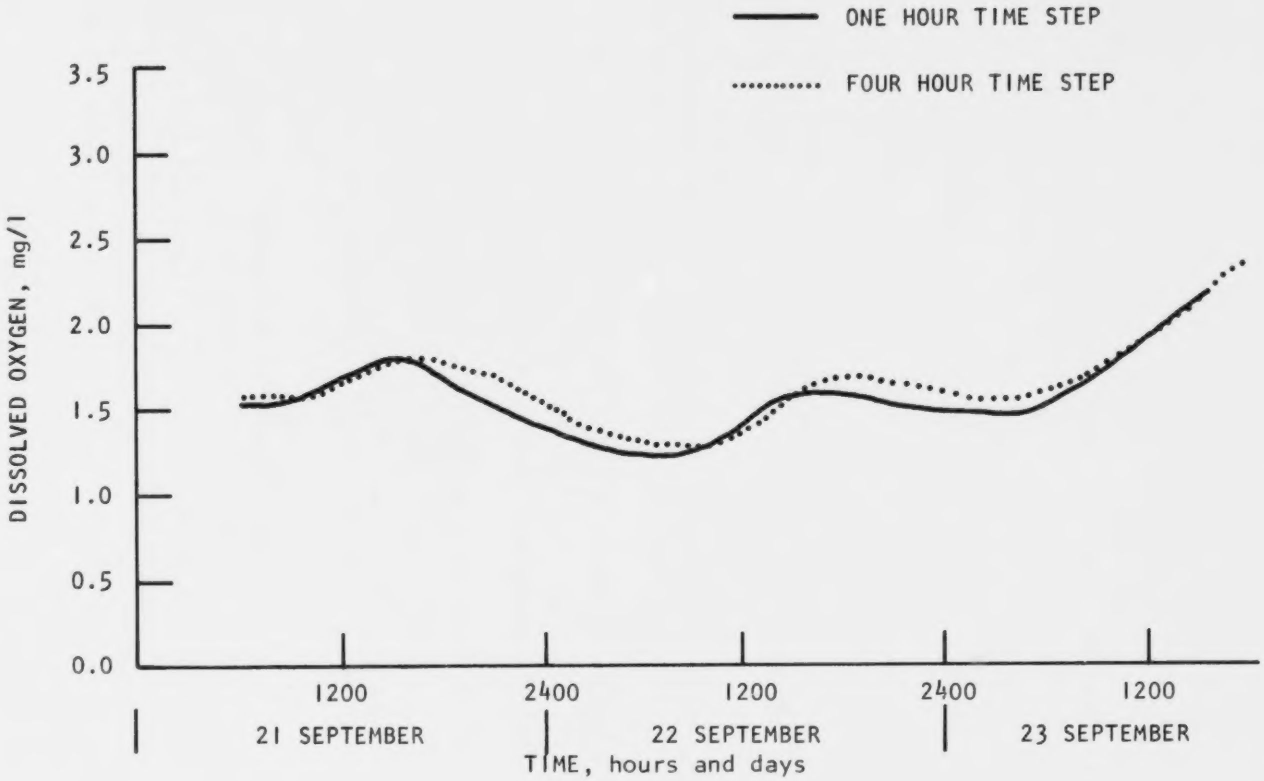


Figure 42. Time step sensitivity analysis for St. Paul Avenue, September 21-23, 1976 (Survey 1).

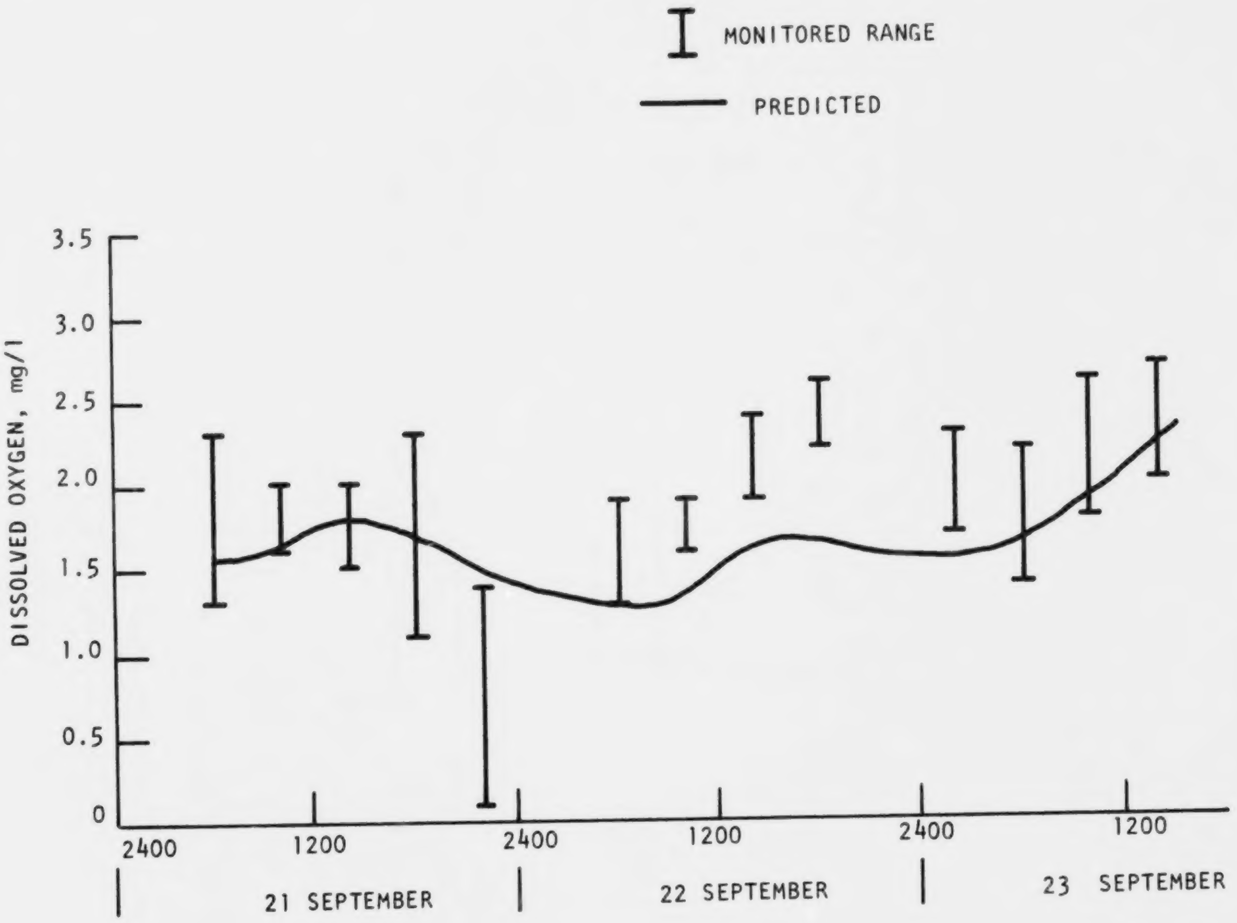


Figure 43. Dry weather verification results for St. Paul Avenue, September 21-23, 1976 (Survey 1).

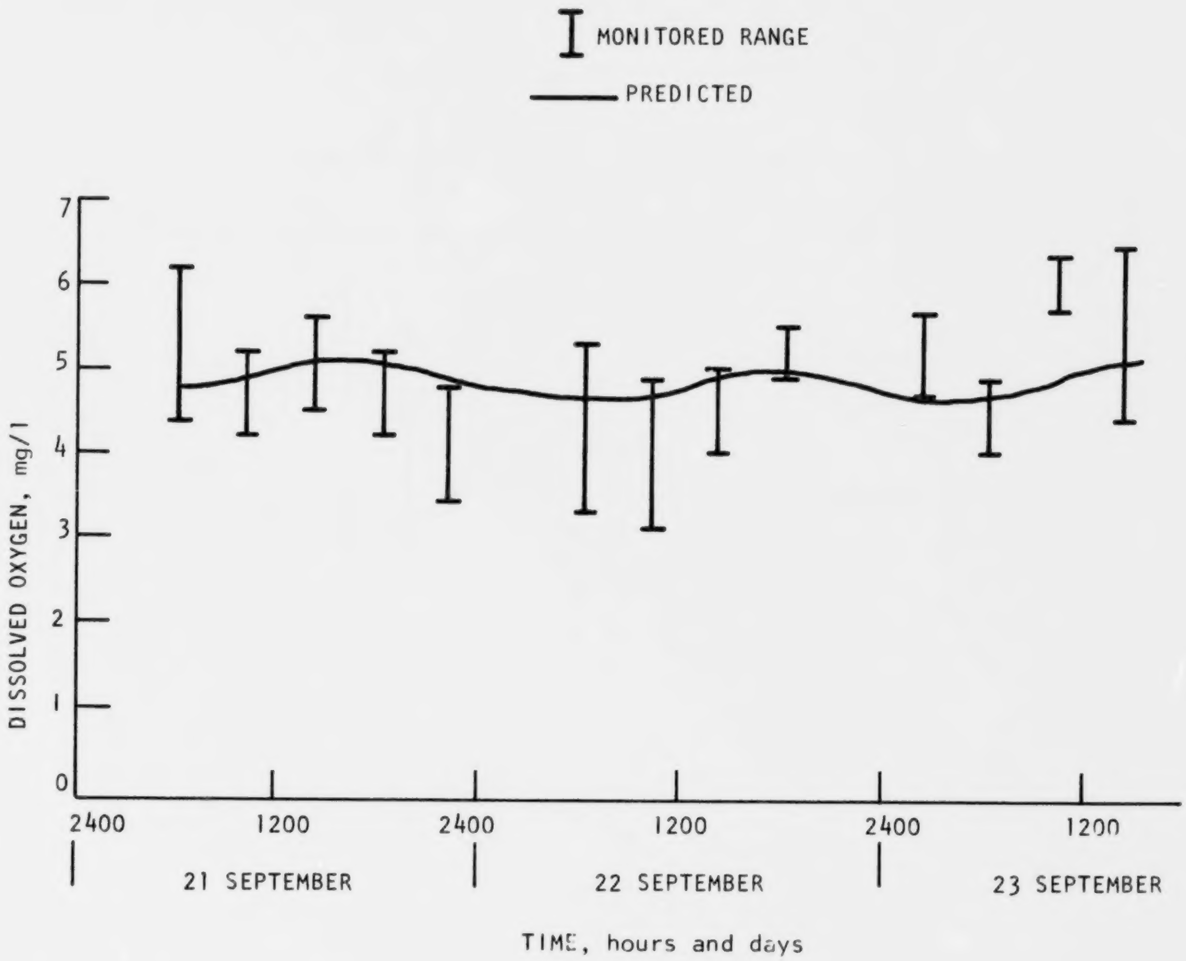


Figure 44. Dry weather verification results for Walnut Street, September 21-23, 1976 (Survey 1).

values illustrates the concentration variations laterally and vertically at the sites. These results indicate that the model provides an accurate estimate of variation in DO in the river under dry weather, steady flow conditions. Predicted values were generally quite close to the observed range at each of the illustrated locations.

The temperature prediction of the model are not quite as accurate as the dissolved oxygen predictions. Figures 45 and 46 illustrate the observed average in the temperature values versus the predicted temperature at the North Avenue Dam and at Capitol Drive. Most of the variation between the observed and predicted values is probably due to the wide range in observed values during each time period. The largest difference between observed averages and predicted values is 3°C which is acceptable for modeling purposes.

Calibration for fecal coliforms with the data from the dry weather survey (Survey I, September, 1976) was not possible. The concentrations observed at all locations at the beginning of the survey were quite substantial. The levels increased further during the first day and then declined (see Figure 16 in Section 4). The only sources of fecal coliforms assumed in this modeling effort were CSO, storm sewers and the upstream load. During dry weather, the only source was the upstream load. Unless fecal coliforms are allowed to have a net growth rate in the river, the observed concentrations cannot be predicted. A net growth rate for fecal coliforms in receiving waters is not feasible. Therefore, fecal coliforms are assumed to be nearly conservative in the river with only a slight die-off rate. This assumption was evaluated with data from the wet weather surveys.

Many of the parameters input to Harper's model were determined from literature values or from the June 1975 calibration. There are a number of parameters which were not measured during either of the intensive surveys which can significantly influence simulation results for dissolved oxygen. These are:

1. Lake Michigan inflow.
2. Phytoplankton concentration.
3. Phytoplankton self-shading factor and settling coefficients.
4. Benthic algae concentration.
5. Base extinction coefficients for light.
6. Sediment oxygen demand.

Periodic sampling for this project has provided additional data on all the above parameters except for the benthic algae concentration, phytoplankton settling coefficient and phytoplankton self-shading factor.

Inflow of Lake Michigan into the Milwaukee River was estimated based upon changes in specific conductance values and total dissolved solids between the North Avenue Dam and the inner harbor. This inflow is quite variable since it is dependent on atmospheric pressure, wind speed and direction and the river flow among other factors. Estimates for this inflow at St. Paul Avenue are between 0 and 150 cfs (4.2 m^3/sec). The

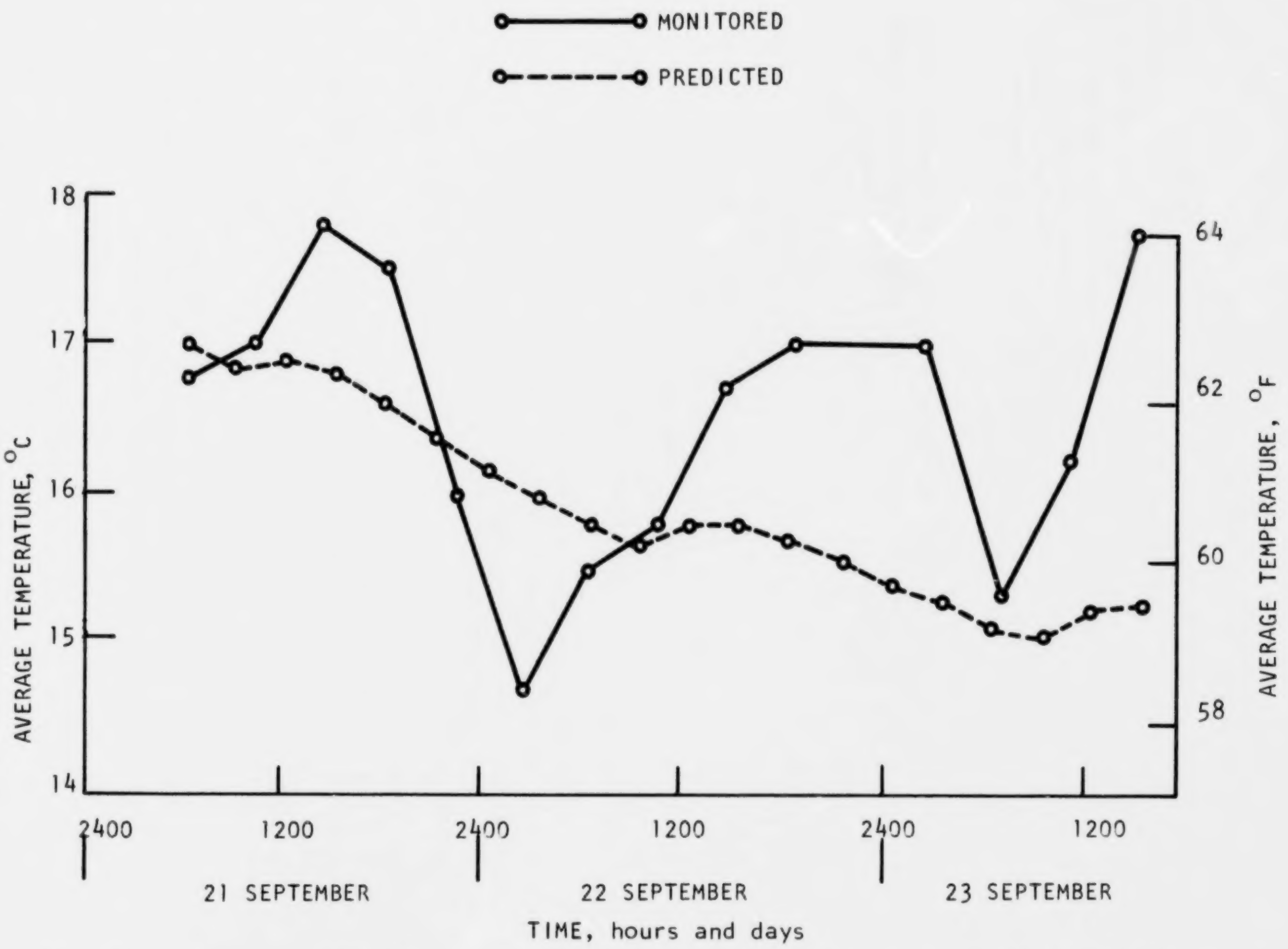


Figure 45. Temperature verification results for North Avenue, September 21-23, 1976 (Survey 1).

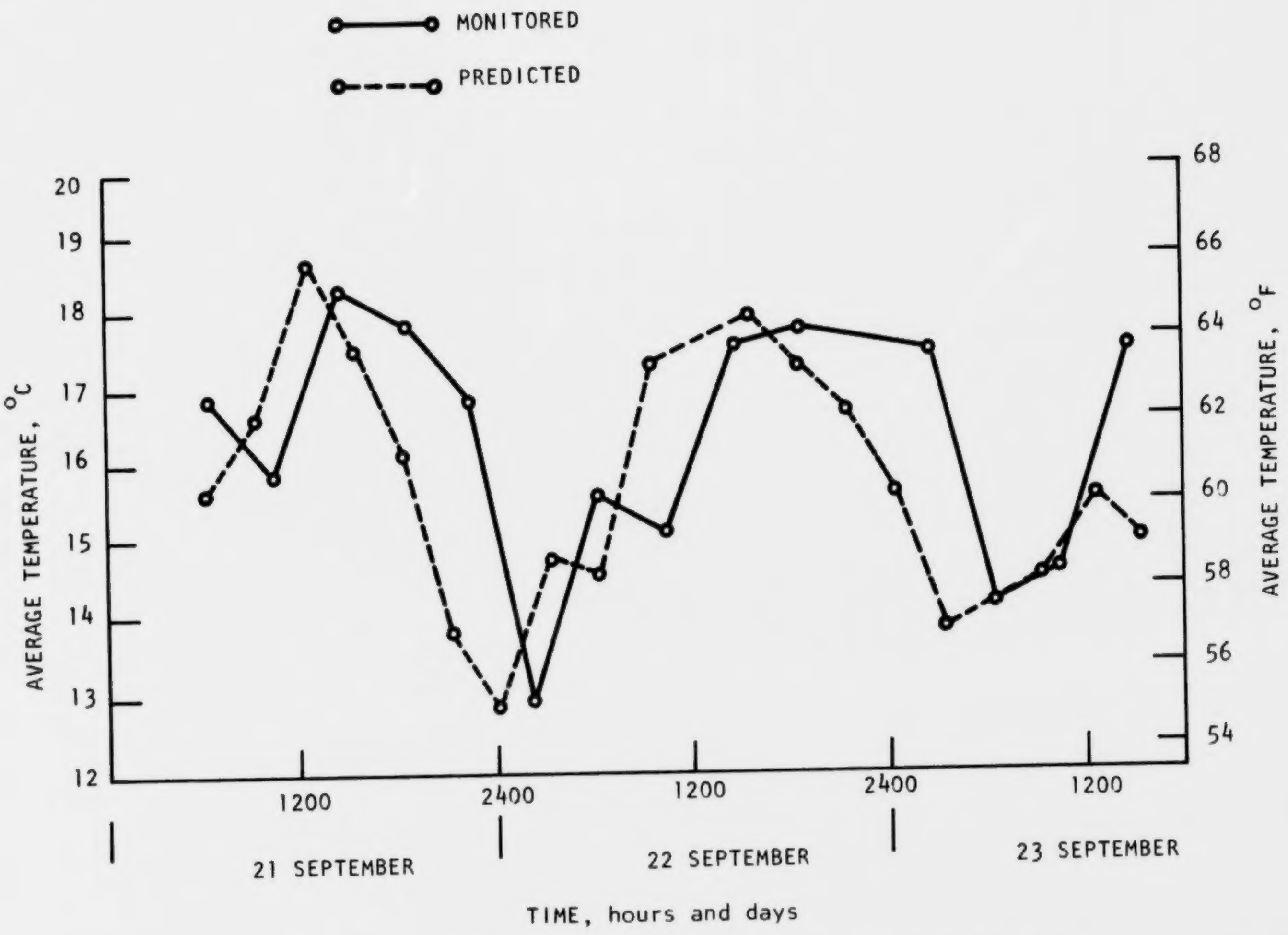


Figure 46. Temperature verification results for Capitol Drive, September 21-23, 1976 (Survey 1).

value utilized in the receiving water model is 55 cfs (1.6 m^3/sec) at the junction of the Milwaukee and Menomonee Rivers. This value was in the range estimated with field data and provides the best match with the calibration data. Estimates of this inflow have also been obtained by integrating velocity measurements taken from several depths (11). An inflow value at St. Paul Avenue of 50 cfs (1.4 m^3/sec) was selected for modeling purposes based upon this analysis.

Periodic sampling for chlorophyll _a_ and measurements of secchi depth during the summer of 1977 were utilized to estimate phytoplankton concentrations and base extinction coefficients input to Harper's model. The results of the sampling for chlorophyll _a_ are listed in Table A-14 in the Appendix. Secchi depth measurements were used to estimate the base extinction coefficients by assuming that this depth is at 10-15 percent of the surface light intensity.

The results of the sediment oxygen demand investigations have provided a range of possible values for each area of the river. SOD rates utilized in Harper's model were calibrated within the observed range. The SOD data is presented in Section 4.

Benthic algae, which are assumed to represent attached algae and macrophytes are only important in the shallow reaches of the Milwaukee River between Port Washington Road and North Avenue. Values for this parameter between 1000 and 8000 mg/m^2 have generally allowed for accurate simulation of DO in the upper portions of the river with the particular concentrations dependent upon the time of year under analysis. The phytoplankton self-shading factor and settling coefficients have been adjusted so that phytoplankton concentration remains in an acceptable range for the time of year being simulated.

Sensitivity analyses have been conducted on a number of parameters in order to determine an acceptable range for wet weather calibration. Figures 47 and 48 illustrate the results of these sensitivity analyses for the base extinction coefficient for light and the sediment oxygen demand rate at St. Paul Avenue in the Milwaukee River for the September 1976 survey. The sensitivity analyses presented represent a ± 0.2/ft (0.06/m) change in the base extinction coefficient and a ± 0.1 gm/m^2-hr change in the SOD rate throughout the modeled portion of the river. Sensitivity analyses performed for this project and calibration results for June 1975 have been utilized to develop the range of input parameter values listed in Table 22. Sensitivity analyses on the size of the time step have indicated that a three hour time step can be utilized for simulations without introducing significant errors.

Wet Weather Calibration/Verification--

The basic methodology employed in calibrating Harper's model for dissolved oxygen during wet weather conditions involved three steps:

1. Determining the CSO and storm sewer loads using the STORM model.

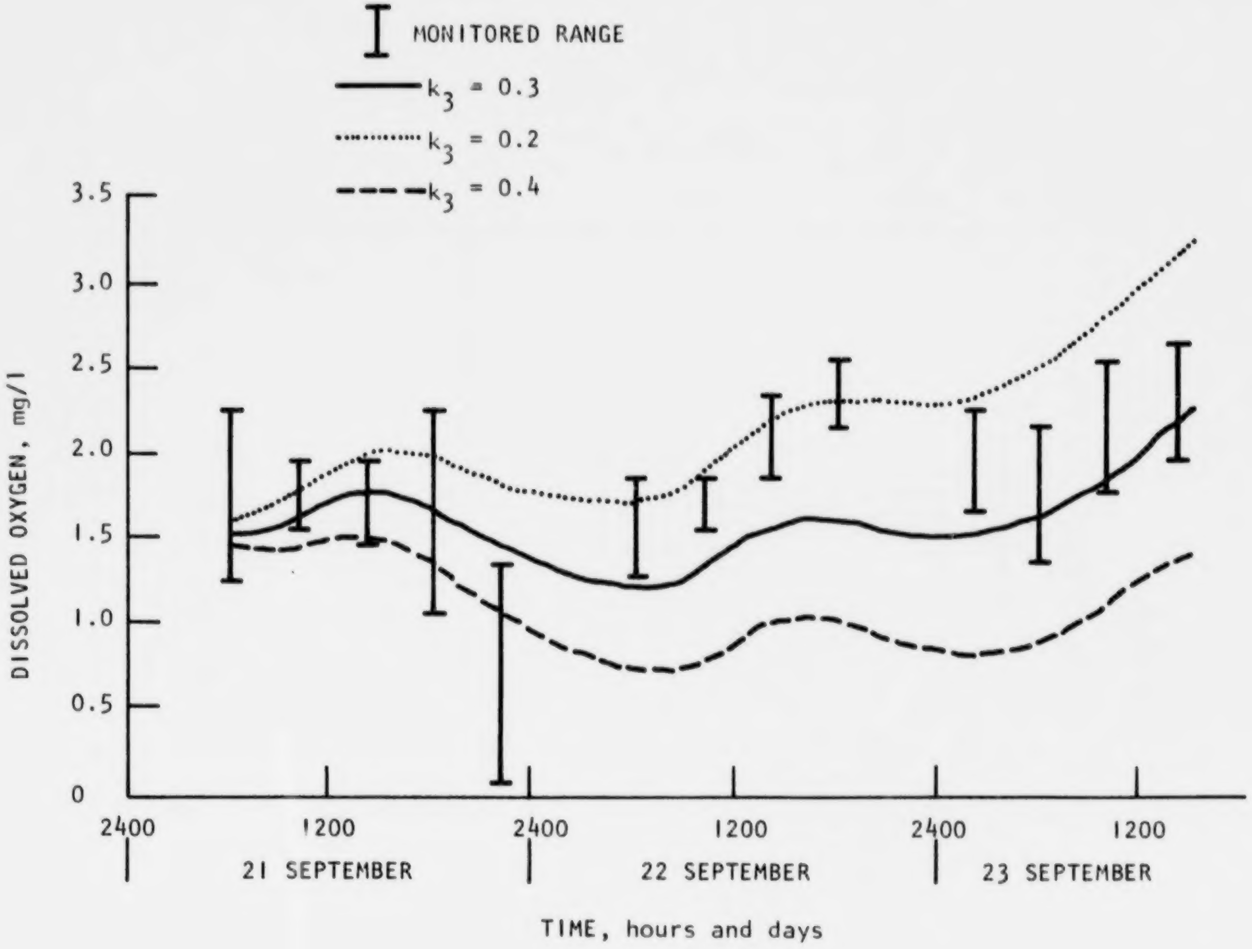


Figure 47. Sensitivity analysis for the dry weather SOD at St. Paul Avenue

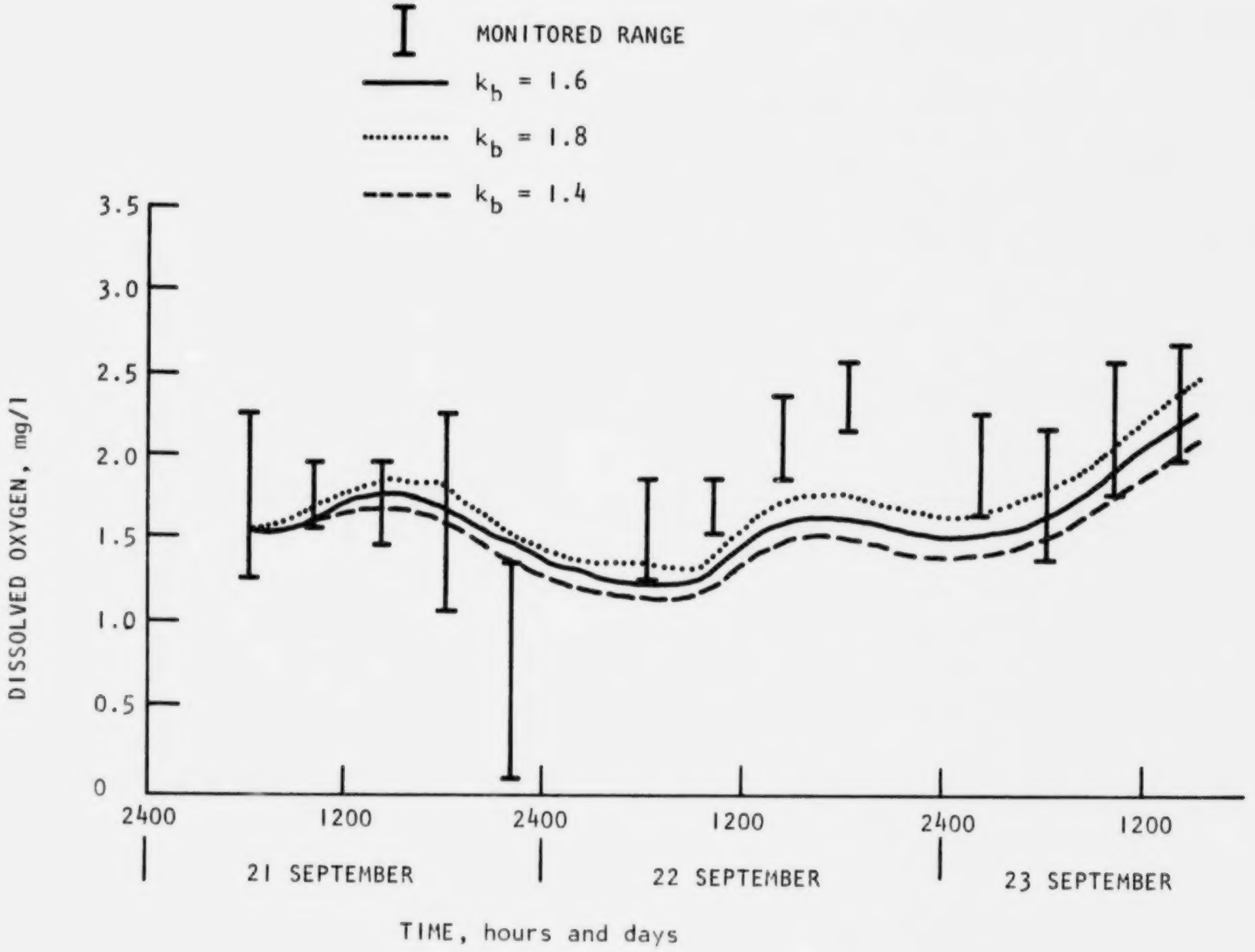


Figure 48. Sensitivity analysis for the base extinction coefficient at St. Paul Avenue.

2. Determining the a, b and T coefficients from the values of ΔDO and the decline duration predicted with the regression equations.

3. Calibrating the model within the acceptable range of values utilized for dry weather calibration/verification.

Data from two intensive surveys of three days duration in May and June, 1977 were utilized to calibrate for wet weather conditions. These intensive surveys were single storm events with the monitoring being initiated immediately after the storm. Three additional surveys in August, 1977, June, 1978 and July, 1978 were used to verify the model. Each of these surveys encompassed multiple rainfall events which made the simulation of CSO impact more difficult.

TABLE 22. RANGE OF INPUT PARAMETER VALUES UTILIZED FOR DRY WEATHER CALIBRATION OF HARPER'S MODEL

Input parameter	Units	Description	Calibration range
k_b	ft^{-1}	Base extinction coefficient	0.5-1.8
K_{phyto}	$(mg/l\text{-}ft)^{-1}$	Phytoplankton self-shading factor	0.01-0.10
U_{02}	mgO_2/mg algae	Production coefficient for algal growth	1.5
k_1	per day	Deoxygenation coefficient	0.05-0.10
k_3	$gm/m^2\text{-}hr$	Sediment oxygen demand	0.0-0.4
K_{CS}	mg/l	BOD half saturation constant for coliform growth	10.0-20.0
k_C	per day	Coliform die-off coefficient	0.0-10.0
Q_L	cfs	Lake Michigan inflow	25-100

m = 0.30xft
m^3/sec = 0.028xcfs

The CSO and storm sewer loadings for each of the storm events utilized in wet weather calibration/verification are listed in Table 23. These loadings were input to the receiving water model by dividing the CSO area tributary to the Milwaukee River into six basins. Each of these basins was modeled separately with STORM and the loads were then assumed to enter this river as six point sources. Due to the relatively large size of the completely mixed longitudinal segments (740 ft - 225 m) utilized in the model for the Milwaukee River, this assumption should introduce no observable errors.

Sensitivity analyses were conducted in order to quantify any error due to these simplifications. This was accomplished by lumping the CSO outfalls into twelve point sources and then eighteeen point sources. Rather than evaluate the differences with a single storm simulation, the differences were evaluated using a one month simulation. The month utilized for this analysis was June and the rainfall input to the STORM model was for June, 1954. This was an extreme month of rainfall with 8.28 inches recorded by the National Weather Service at Mitchell Field in Milwaukee. The differences observed with the various number of point sources simulating CSO outfalls was evaluated using the area below a DO value of 5.0 mg/l and above the predicted levels of DO. This area is actually the magnitude in mg-day/l of the time the DO is below the level. Table 24 lists the results of this analysis for several sites in the study area of the Milwaukee River. The largest effect of the lumping of CSO outfalls was observed at Walnut Street. By using eighteen point sources rather than six distributed along the river, the magnitude of the area below 5.0 mg/l increased by 5 percent. The variations at the North Avenue, Wells Street and St. Paul Avenue sites were much less. This analysis indicates that the errors introduced by the lumping into only six point sources does not significantly effect model results.

Calibration of Harper's model for the wet weather events mainly involved determining a consistent method of obtaining the a and b coefficients from the predicted dissolved oxygen decline and decline duration. The time varying SOD equation provides an exponential decay of the wet weather SOD rate, therefore, it is possible to adjust the a and b coefficients from the extreme condition of nearly all the oxygen demand being exerted in the first three hour time step to a nearly linear exertion of the demand over many time steps.

The observed rapid decline in dissolved oxygen following a runoff event can generally be approximated as a linear decline. On the basis of this observation, the a and b coefficients have been determined so that the decline is as linear as possible. This was accomplished by adjusting the coefficients for a given storm event until the average SOD during the predicted decline period matched the predicted decline in dissolved oxygen. The coefficients were then further adjusted until the peak SOD rate during the first three hour time step was between 1.0 and 1.5 times the average SOD rate. Storm events with large predicted ΔDO values had the highest ratios of peak to average SOD rate.

This methodology proved to be extremely successful for calibration of Harper's model for wet weather conditions. Figure 49 through 52 illustrate the calibration results for the surveys conducted in May and June, 1977 (Surveys 2 and 3), at several locations in the Milwaukee River. The May 31, 1977 storm event (Survey 2) was a small volume high intensity storm with 0.17 in. (0.43 cm) of rain recorded during a 25 minute period at the Broadway Street precipitation gauge. This small event had a severe impact on DO concentrations particularly at the Wells Street site. These low DO concentrations appear to be due to three factors:

TABLE 23. CSO AND SEPARATE STORM SEWER LOADINGS TO THE MILWAUKEE RIVER FOR THE INTENSIVE MONITORING SURVEYS PREDICTED WITH STORM

		CSO loadings				Separate storm loadings			
Survey number	Initiation date	Overflow volume, mil. gal.	Suspended solids, lbx10^3	CBOD, lbx10^3	Fecal coliforms, counts	Runoff volume, mil. gal.	Suspended solids, lbx10^3	CBOD, lbx10^3	Fecal coliforms, counts
1	9-21-76	-	-	-	-	-	-	-	-
2	5-31-77	5.4	34	17	4.0×10^{15}	8.4	3	5	3.1×10^{12}
3	6-17-77	69.2	178	52	7.7×10^{15}	167.6	115	30	1.2×10^{13}
4	8- 3-77	97.5	165	101	1.9×10^{16}	245.2	128	62	3.3×10^{13}
5	6-16-78	209.3	549	210	3.1×10^{16}	573.9	544	158	7.1×10^{13}
6	7-26-78	54.7	122	64	1.1×10^{16}	129.3	89	42	2.2×10^{13}

m^3 = 0.038xgal.
kg = 0.454xlb.

TABLE 24. SENSITIVITY ANALYSIS OF LUMPING OF CSO OUTFALLS INTO POINT SOURCES IN RECEIVING WATER MODEL

		Magnitude below 5.0 mg/l, mg-days/l		
		Number of point sources		
Site	Distance upstream, mi (km)	6	12	18
St. Paul Avenue	0.2 (0.3)	113.5	115.5	116.9
Wells Street	0.6 (1.0)	88.7	91.8	93.9
Walnut Street	1.4 (2.3)	21.1	21.7	22.2
North Avenue	2.4 (3.9)	0.0	0.0	0.0

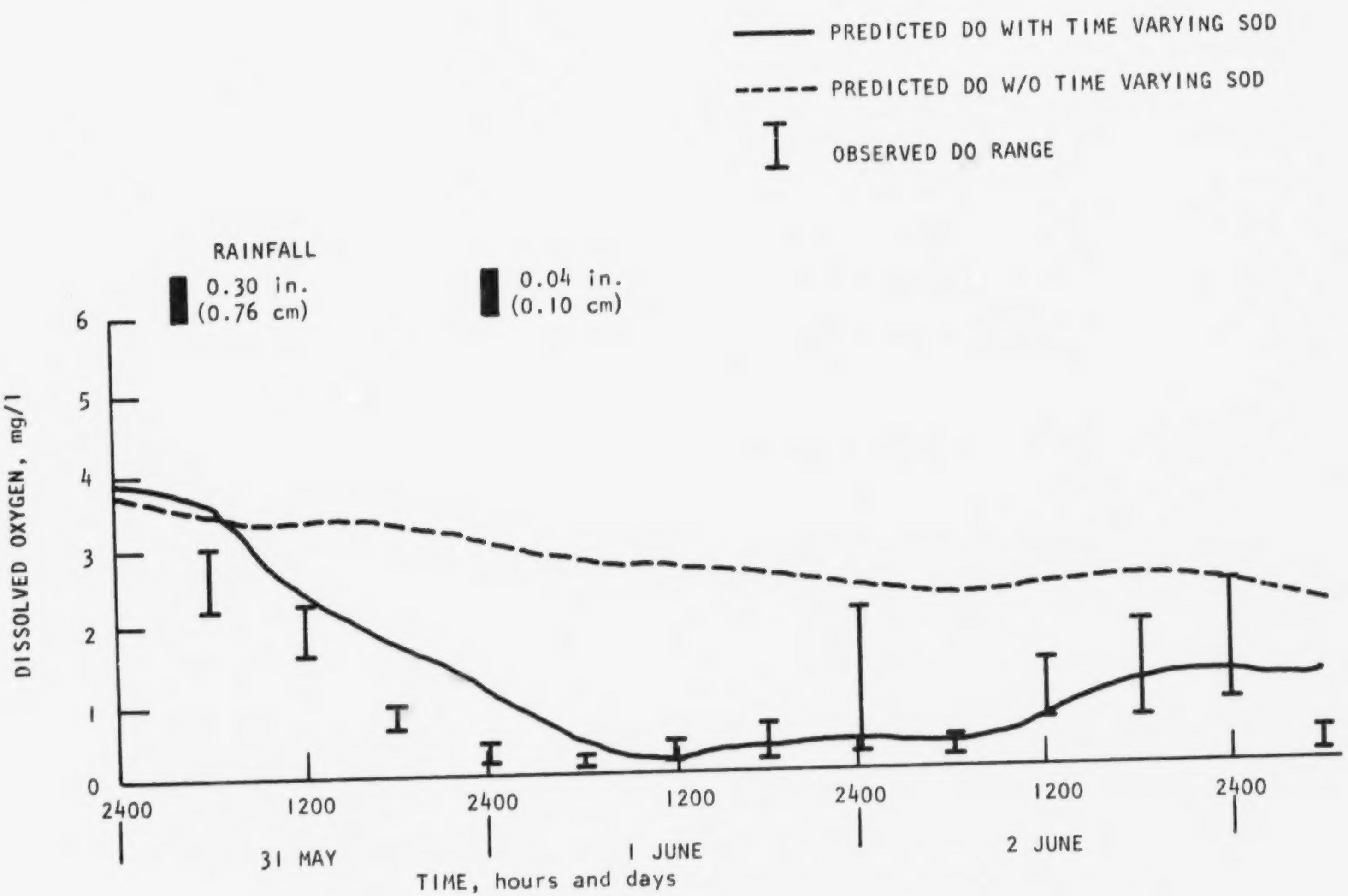


Figure 49. Wet weather calibration results for Wells Street, May 31 - June 3, 1977 (Survey 2).

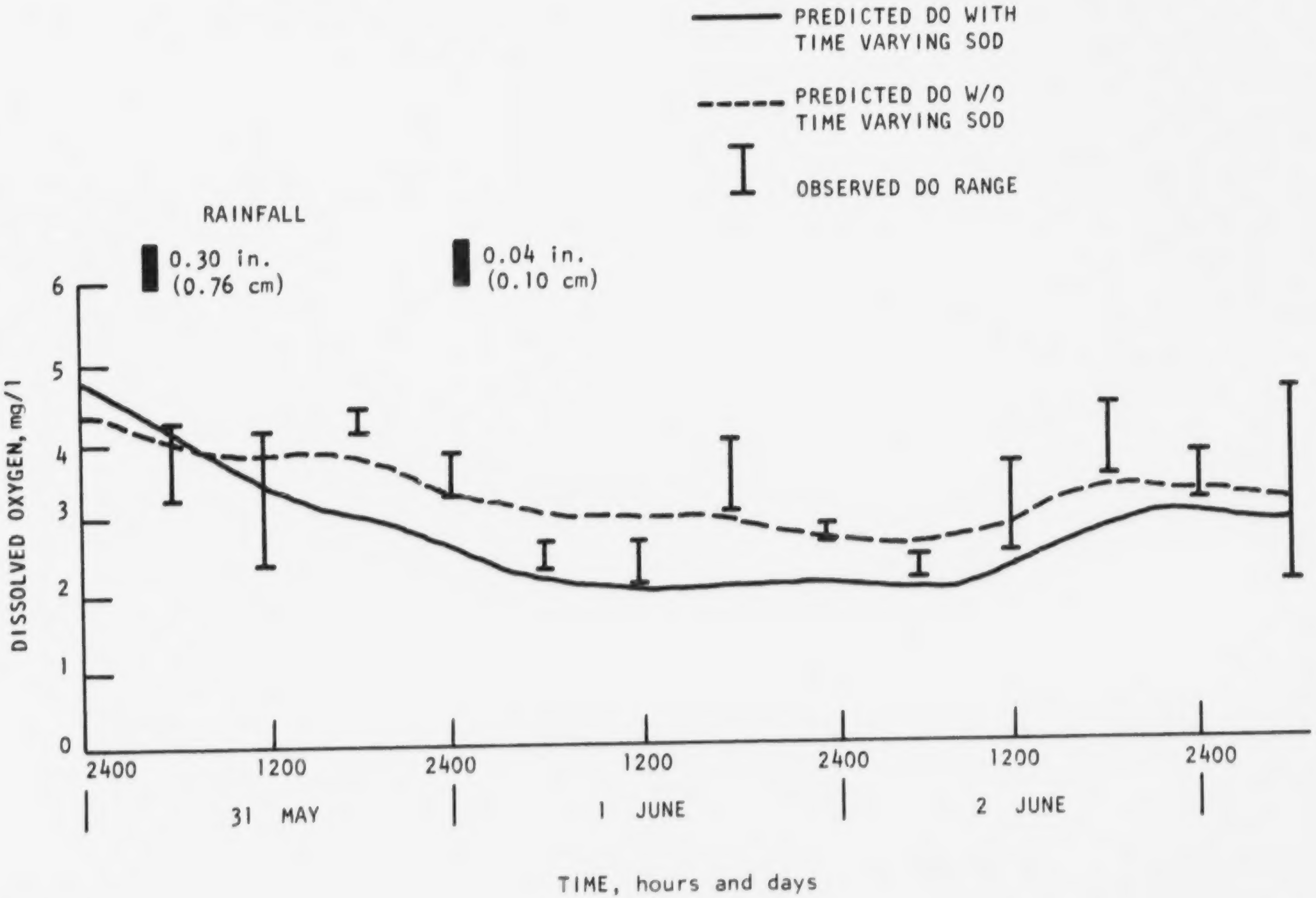


Figure 50. Wet weather calibration results for Walnut Street, May 31 - June 3, 1977 (Survey 2).

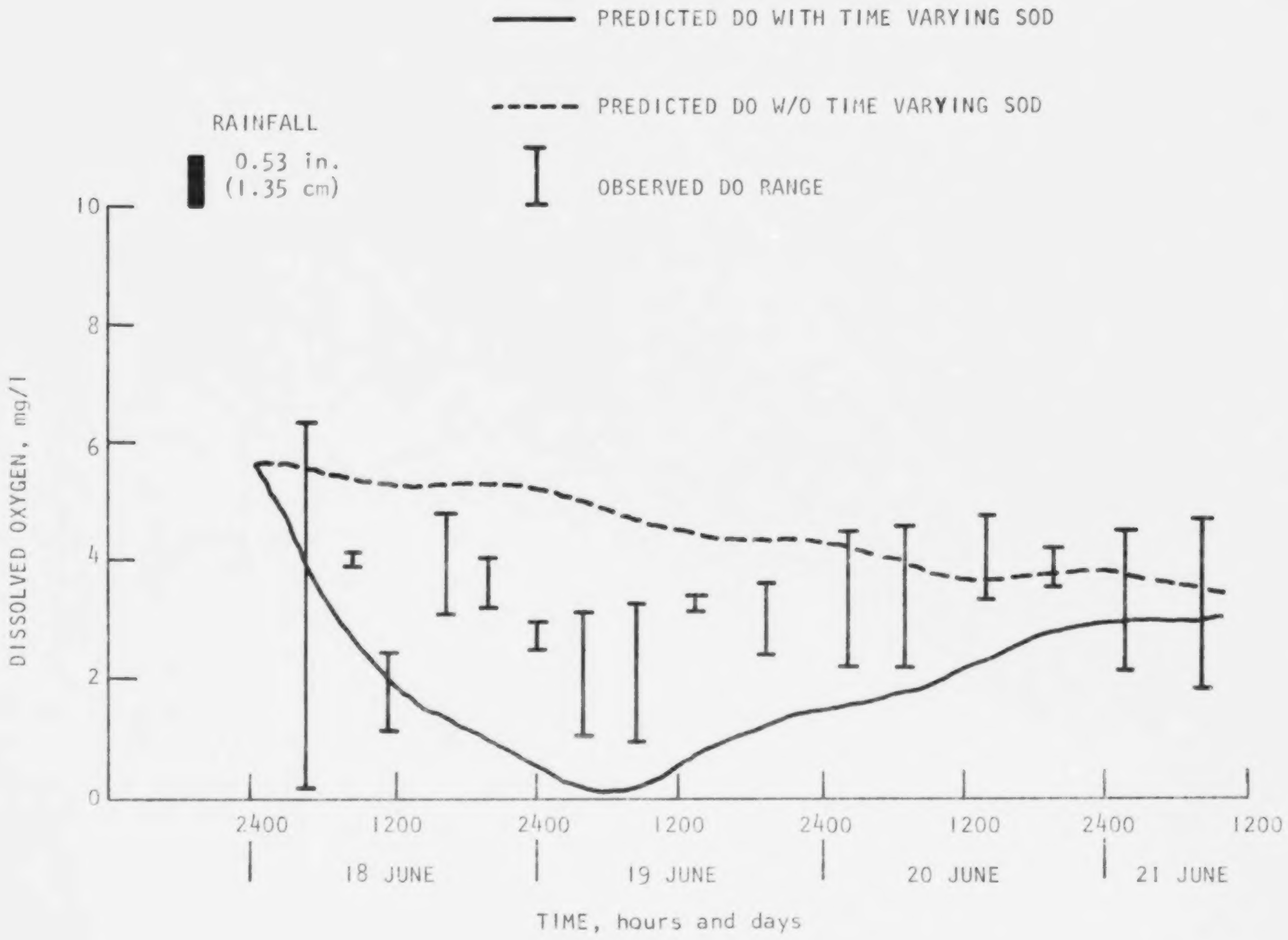


Figure 51. Wet weather calibration results for St. Paul Avenue, June 18-21, 1977 (Survey 3).

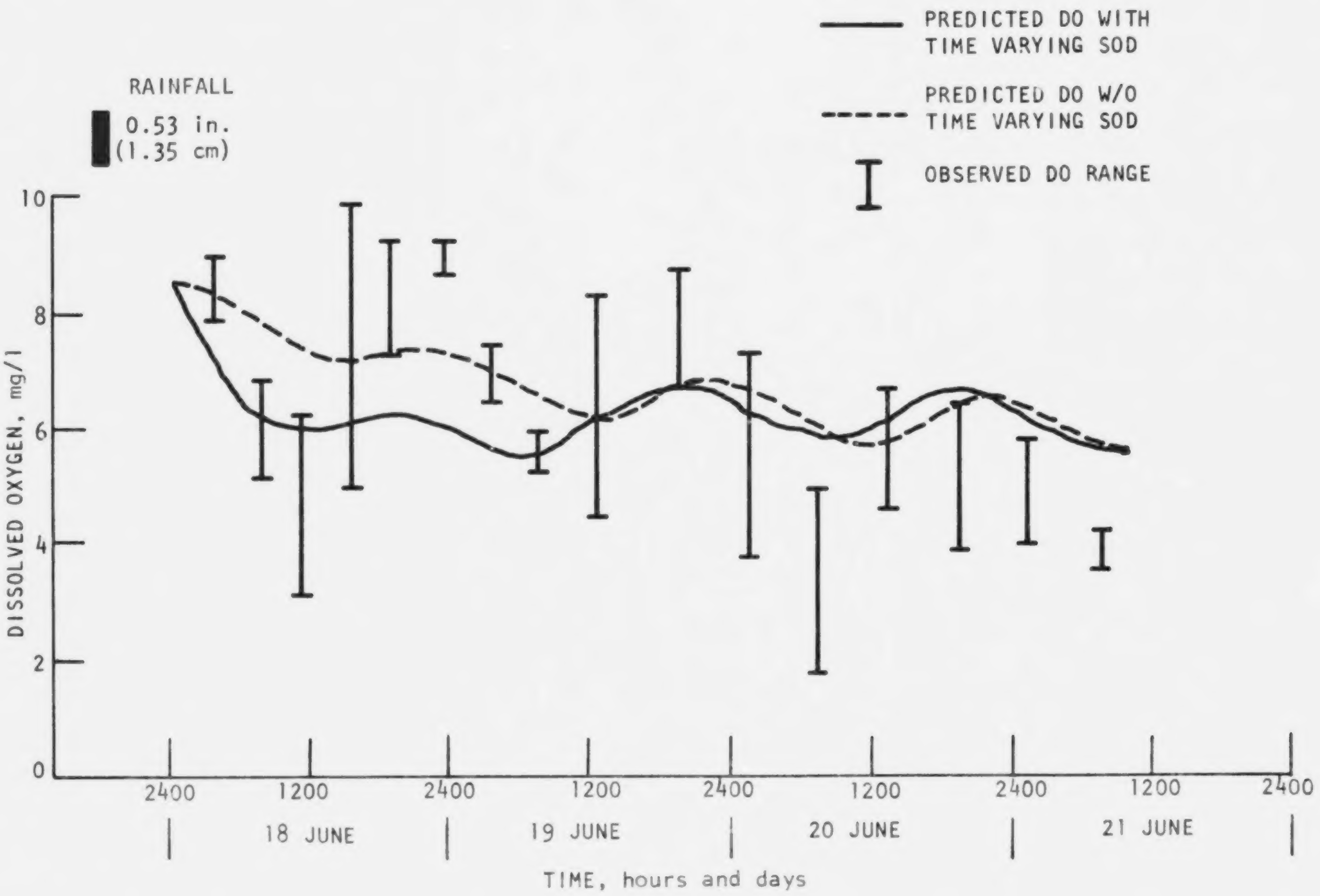


Figure 52. Wet weather calibration results for Walnut Street, June 18-21, 1977 (Survey 3).

1. The low river flow - 100 cfs.

2. A large load of pollutants due to more than a month of dry weather proceeding the storm.

3. Sediment scour from the submerged CSO outfalls in the lower Milwaukee River.

Using the input parameters determined in the dry weather calibration/verification and the a and b coefficients estimated as described previously, the instream model provided a good prediction of observed DO concentrations at most of the monitoring locations.

In order to demonstrate the sensitivity of the model to the time varying SOD equation and the methodology utilized to adjust the a and b coefficients, all of the calibration/verification figures show the predicted curve if only the dry weather SOD rate and soluble (and suspended) load is considered. It is obvious from this comparison that the significance of this equation on model output varies from survey to survey on the basis of the rainfall volume, CSO volume and river flow. These results will be utilized in Section 7 to evaluate the impact of CSO on DO in the Milwaukee River.

The model predictions for the June 17, 1977 survey (Survey 3) are not as close to the observed values as those for the May wet weather survey (Figures 51 and 52). The DO concentrations predicted by the model recover at a faster rate than those observed during the survey. The results are still quite acceptable for model calibration purposes.

Table 25 is a list of the calibrated input parameters used for Harper's model. The only parameters that were assumed to vary throughout a runoff year (April through October) were the phytoplankton concentrations and the benthic algae concentrations.

In order to verify the model it was necessary to determine the accuracy of the model prediction after a series of storm events. The August 3-8, 1977 survey (Survey 4) was utilized for the verification since there were four separate storm events during the first three days of the five day survey. Figures 53 and 54 and Appendix Figures A-11 and A-12 illustrate the results of this verification run. Harper's model accurately predicts the rapid decline following the initial storm event at 2200 on August 3, 1977 and the dissolved oxygen recovery beginning on August 6, 1977. The rapid recovery is partially a function of DO production in the river by phytoplankton. The change in river flow from 105 cfs at the beginning of the survey to about 450 cfs at the end of the survey as observed at the Estabrook Park USGS gauging station also accounts for a portion of the DO recovery.

Figures 55 through 59 illustrate the verification results for the June 1978 (Survey 5) and July 1978 (Survey 6) wet weather surveys. The DO results at

TABLE 25. CALIBRATION VALUES FOR HARPER'S MODEL INPUT PARAMETERS FOR THE MILWAUKEE RIVER

River reach	Input parameter	Units	Description	Calibration value
Brown Deer Road to North Avenue	k_b	ft^{-1}	Base extinction coefficient	0.9
North Avenue to Walnut Street	k_b	ft^{-1}	Base extinction coefficient	1.4
Walnut Street to River Junction	k_b	ft^{-1}	Base extinction coefficient	1.6
All	K_{phyto}	$(mg/l\text{-}ft)^{-1}$	Phytoplankton self-shading factor	0.05
All	U_{02}	mgO_2/mg algae	Production coefficient for algal growth	1.5
Brown Deer Road to Port Washington Road	k_1	per day	Deoxygenation coefficient	0.05
Port Washington Road to River Junction	k_1	per day	Deoxygenation coefficient	0.10
Brown Deer Road to Port Washington Road	k_3	gm/m^2-hr	Sediment oxygen demand	0.10
Port Washington Road to North Avenue Dam	k_3	gm/m^2-hr	Sediment oxygen demand	0.20
North Avenue Dam to River Junction	k_3	gm/m^2-hr	Sediment oxygen demand	0.30
All	K_{CS}	mg/l	BOD half saturation constant for coliforms	20.0
All	k_C	per day	Coliform die-off coefficient	5.0
River Junction	Q_L	cfs	Lake Michigan inflow	55.0

m = 0.30xft.
m^3/sec = 0.28xcfs

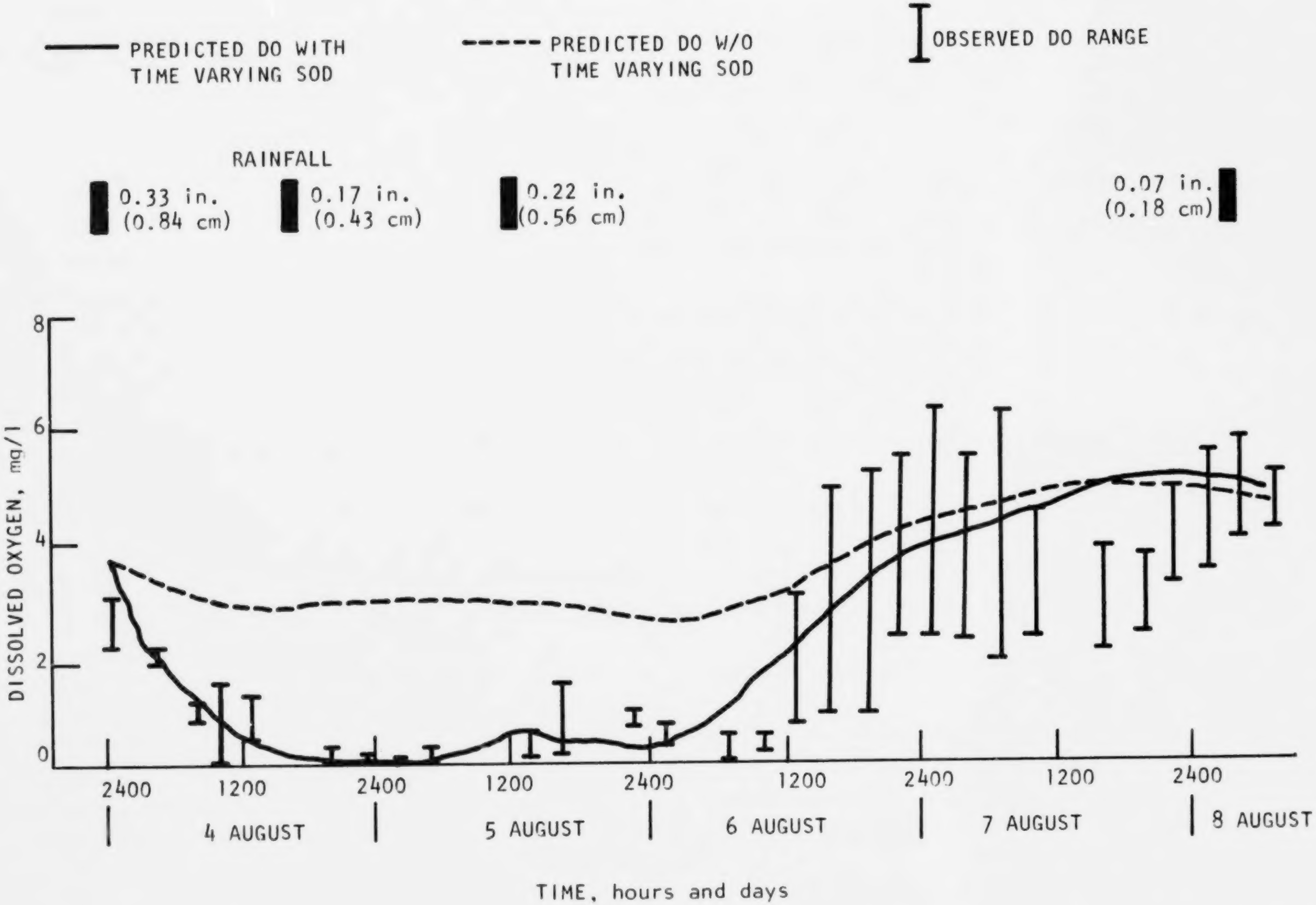


Figure 53. Wet weather verification results for St. Paul Avenue, August 4-8, 1977 (Survey 4).

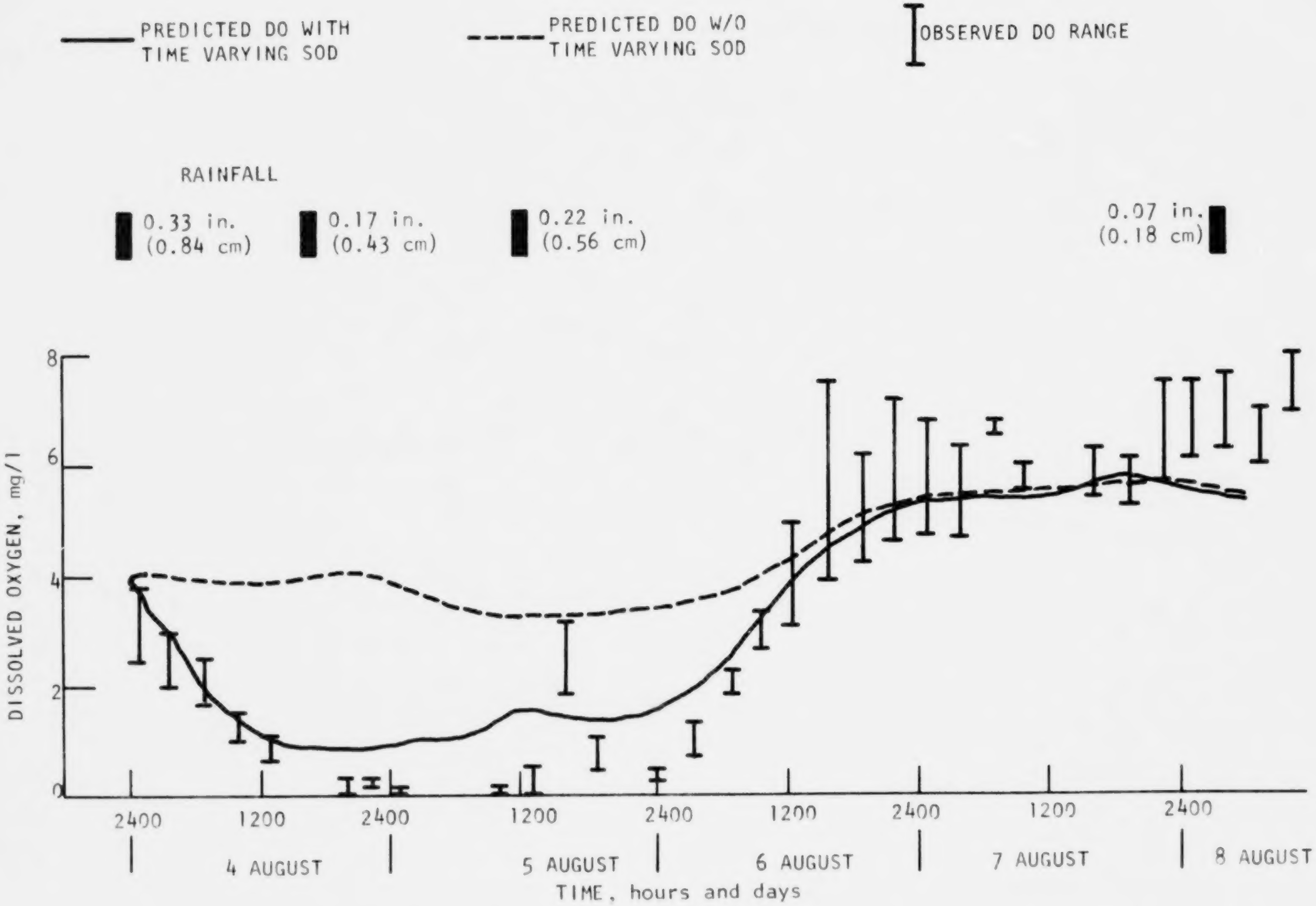


Figure 54. Wet weather verification results for Wells Street, August 4-8, 1977 (Survey 4).

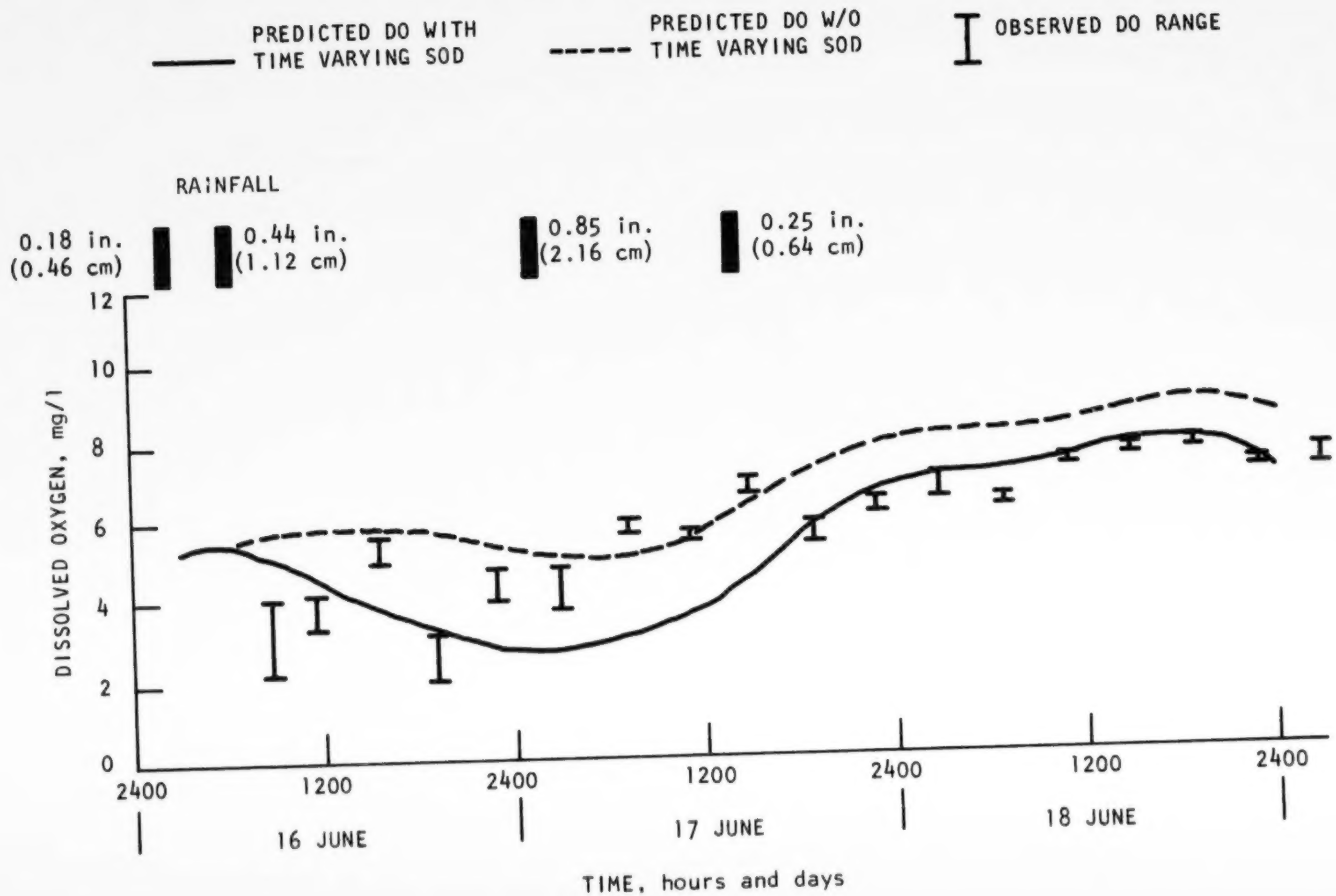


Figure 55. Wet weather verification results for St. Paul Avenue, June 16-18, 1978 (Survey 5).

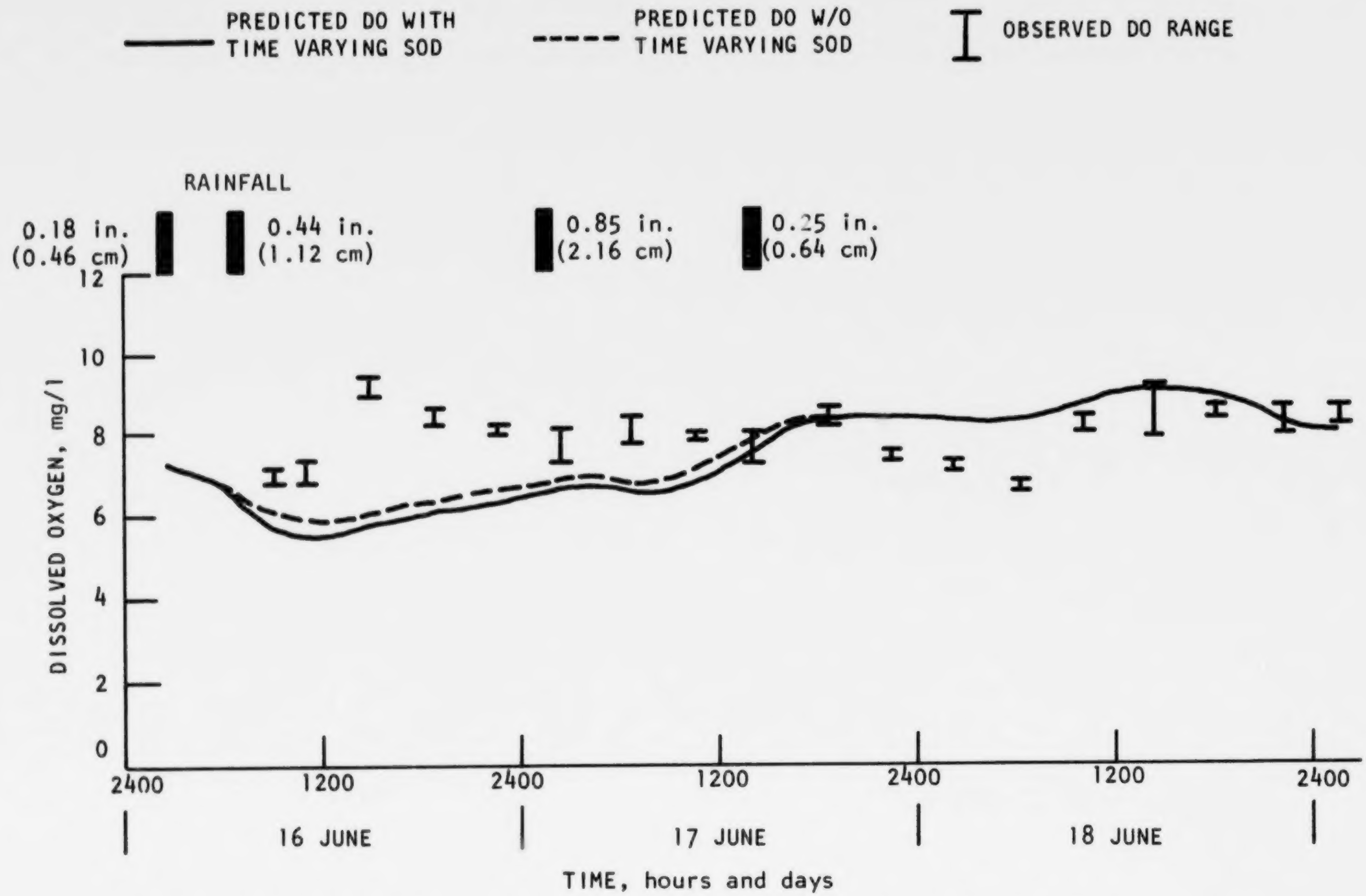


Figure 56. Wet weather verification results for Walnut Street, June 16-18, 1978 (Survey 5).

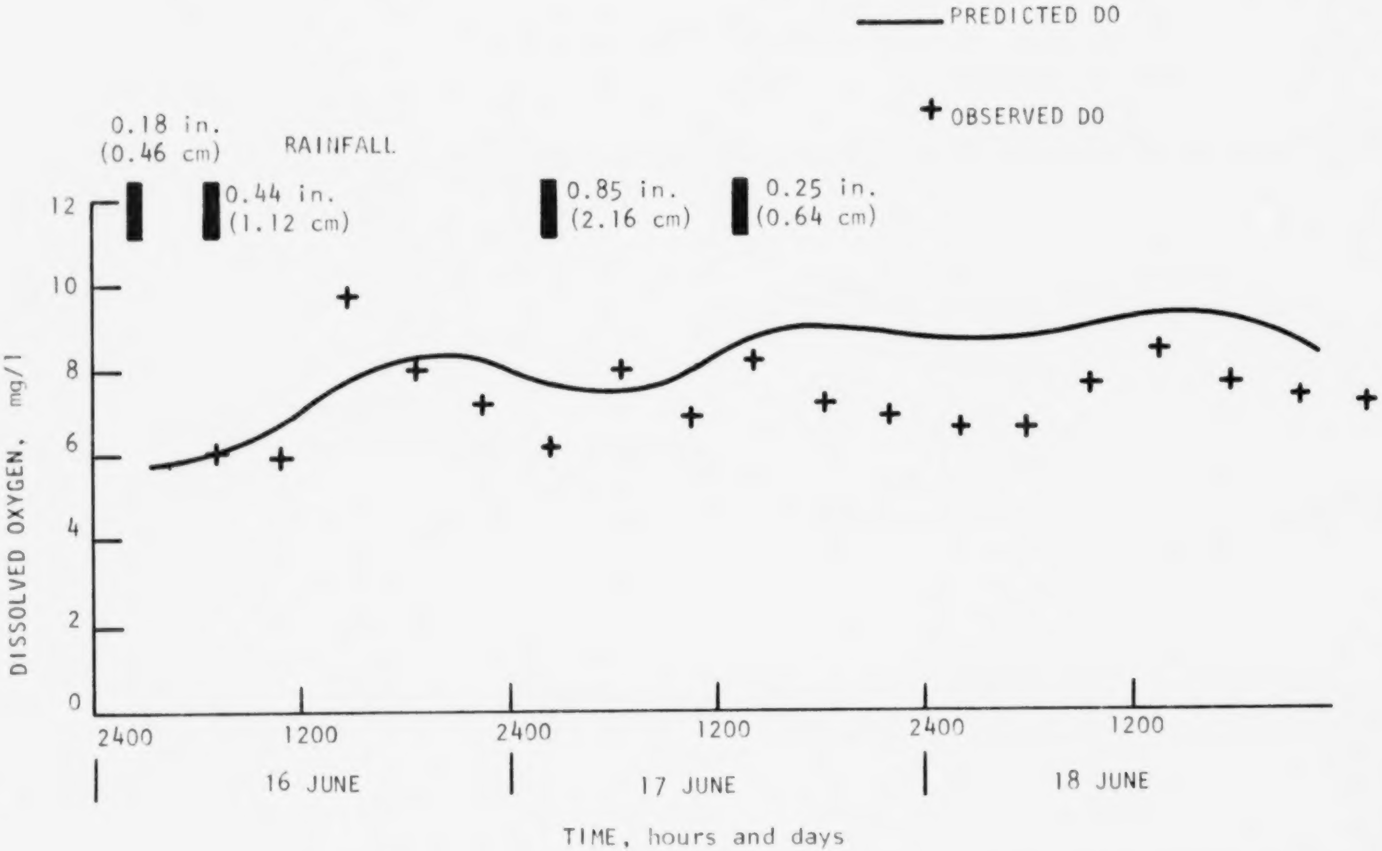


Figure 57. Wet weather verification results for North Avenue, June 16-18, 1978 (Survey 5).

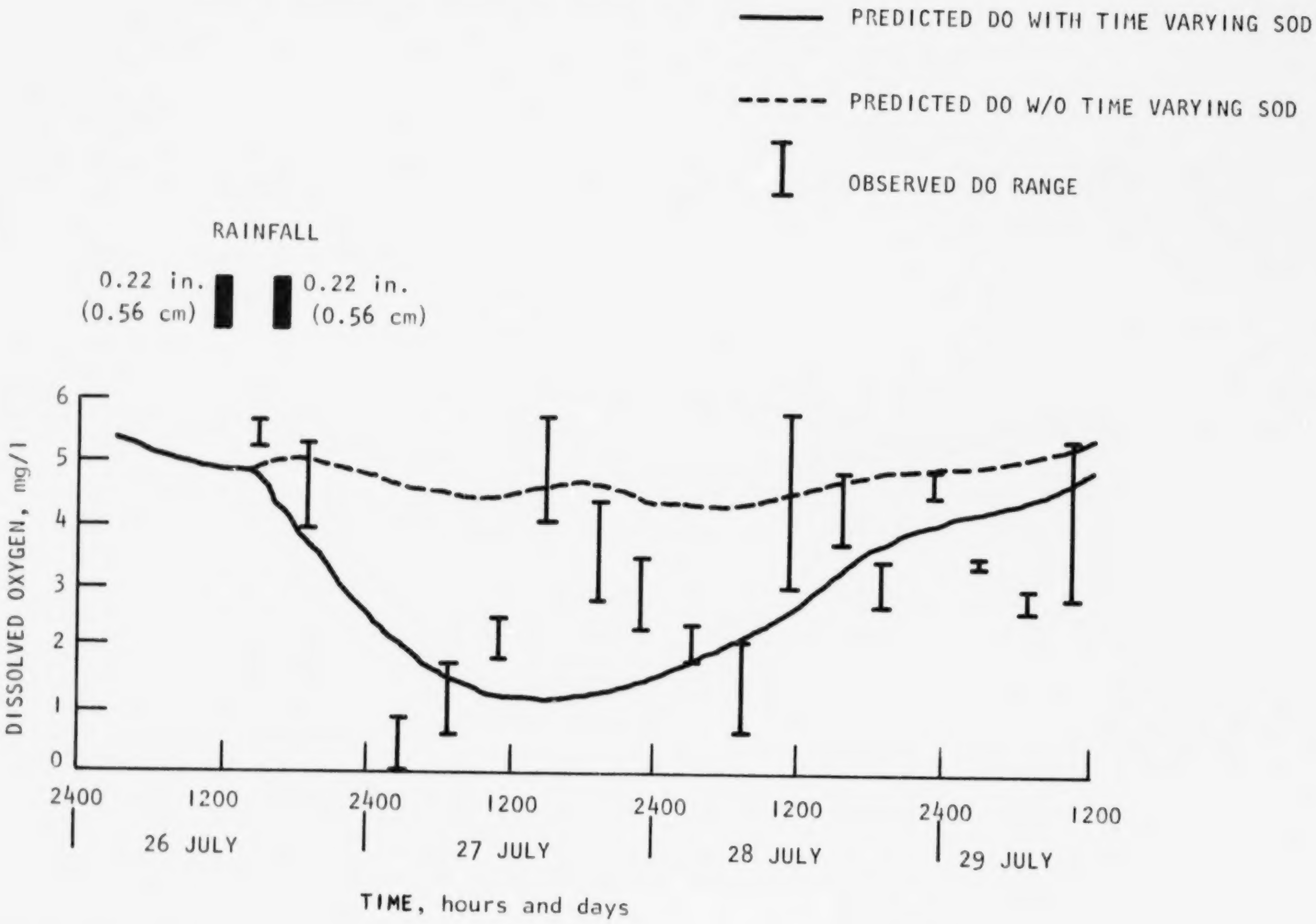


Figure 58. Wet weather verification results for St. Paul Avenue, July 26-29, 1978 (Survey 6).

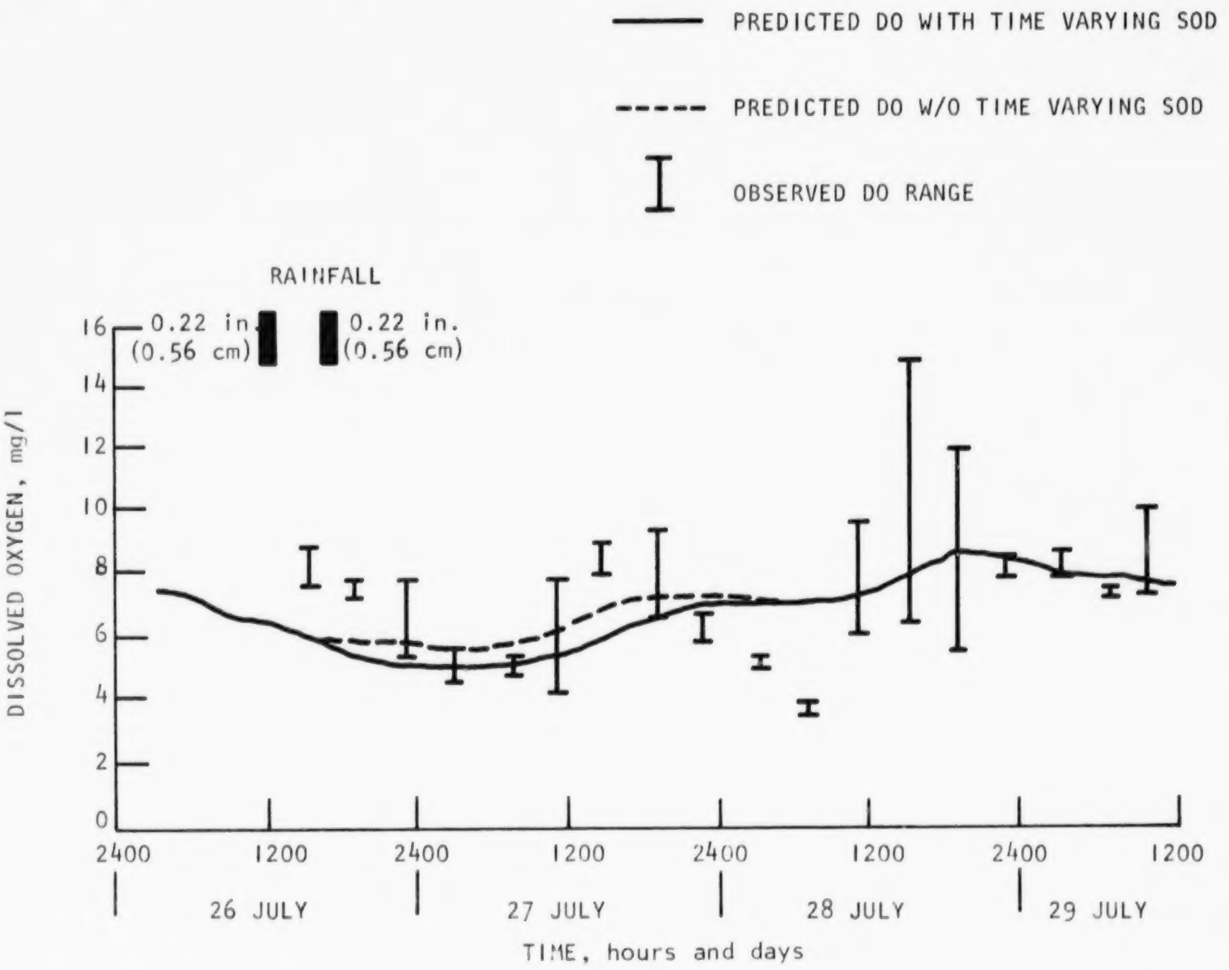


Figure 59. Wet weather verification results for Walnut Street, July 26-29, 1978 (Survey 6).

St. Paul Avenue for Survey 5 (Figure 55) indicate that the model predicts fairly well for this event. The observed DO values are quite inconsistent during the initial day of the survey which accounts for some of the differences between observed and predicted values. The predicted DO does not recover quite as rapidly as the observed DO, however, the general fit of predicted to observed values is good. The predictions at the Walnut Street and North Avenue sites (Figures 56 and 57) are also quite good for this survey. During Survey 6, a marked diurnal variation in DO was observed even in the lower portions of the river. The general trend in DO levels was matched at the St. Paul Avenue site (Figure 58). However, the diurnal variation was not predicted well. Variations of this magnitude were not observed at this station during other surveys or from the continuous monitoring results for similar times of the year. Unrealistically high values of phytoplankton would need to be input to the model to match the observed values. The verification results for DO at the Walnut Street (Figure 59) and North Avenue (Appendix Figure A-15) sites are much better. Despite some of the variability between observed and predicted values, the model predictions are adequate to verify the model.

A source of error for all of the calibration/verification simulations was the assumption of a constant inflow from Lake Michigan. This inflow was considered a constant during all river flow conditions which may account for some of the variation between monitored and predicted values. The inflow only has a measureable impact on simulation results during low flow conditions. During high flows, the inflow was still simulated, however, the impact on predicted values was negligible. Although the model predictions are generally good, accurate simulations of the Lake-river interchange during all flow conditions will only be possible if a relationship between inflow and river flow (or another parameter) can be developed.

As mentioned in the discussion of dry weather calibration, accurate predictions of current levels of fecal coliforms was impossible due to the assumptions utilized in determining the loadings from the storm sewer areas. At the present time, there are numerous cross-connected storm sewers and some combined sewers which discharge into Lincoln Creek. There are also some cross-connected storm sewers discharging into the Milwaukee River upstream of the CSO area. There is an on-going program to eliminate these discharges. The STORM model was calibrated for these areas under the assumptions that all the cross-connections will be eliminated and that the combined sewers discharging to Lincoln Creek will also be eliminated.

Despite the assumptions utilized for the storm sewer loadings, the fecal coliform concentrations predicted for monitoring sites in the CSO area of the Milwaukee River are fairly good. Figures 60 through 62 illustrate the model predictions of fecal coliform concentrations versus the monitoring results. Figure 62 illustrates the predicted versus observed values at Port Washington Road during the August 3-8, 1977 wet weather survey. This site is directly downstream from the junction of Lincoln Creek and the Milwaukee River. It is obvious from the illustration that the actual load from Lincoln Creek is much higher than the predicted load.

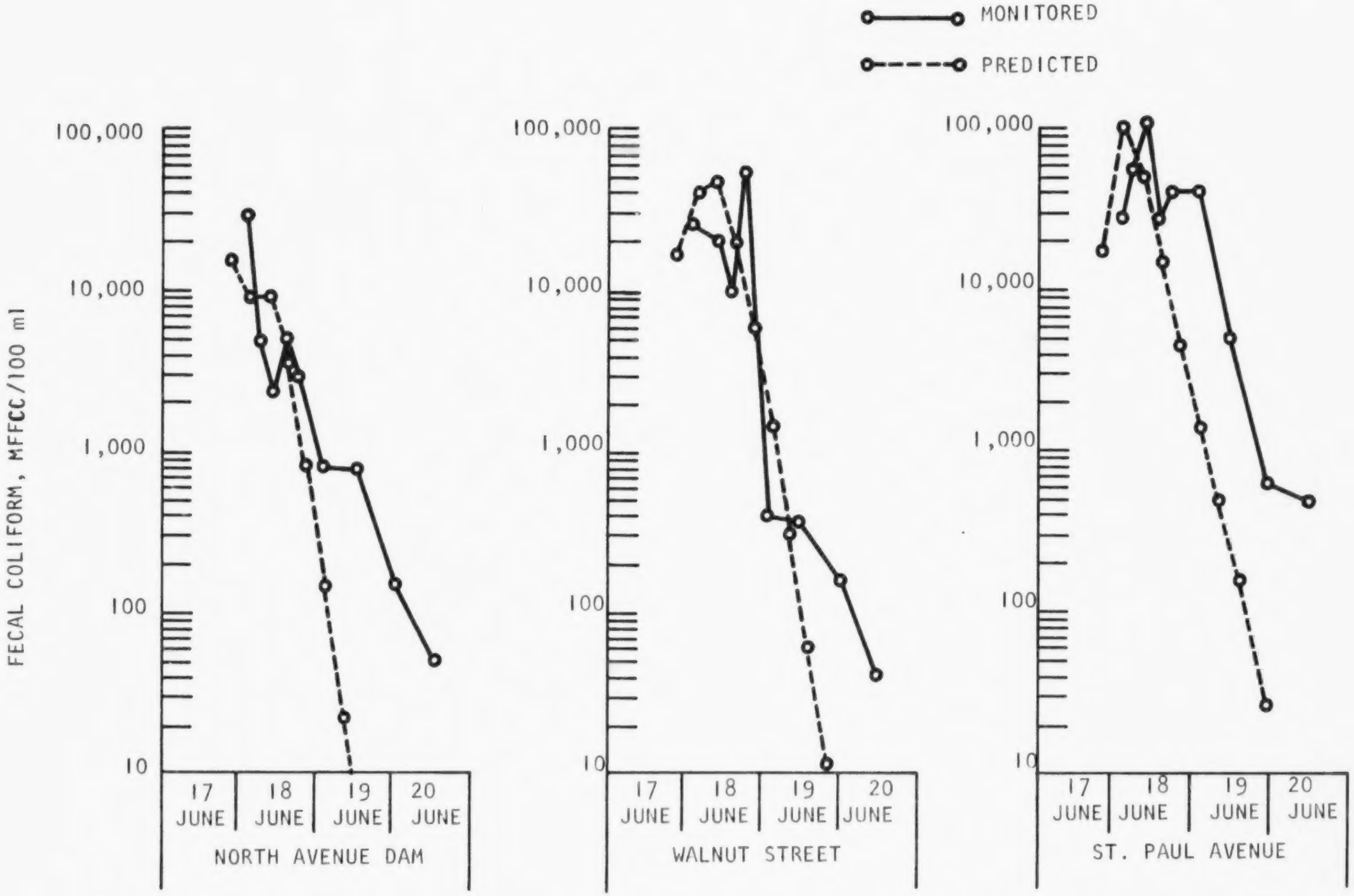


Figure 60. Wet weather fecal coliform calibration results, June 18-20, 1977 (Survey 3).

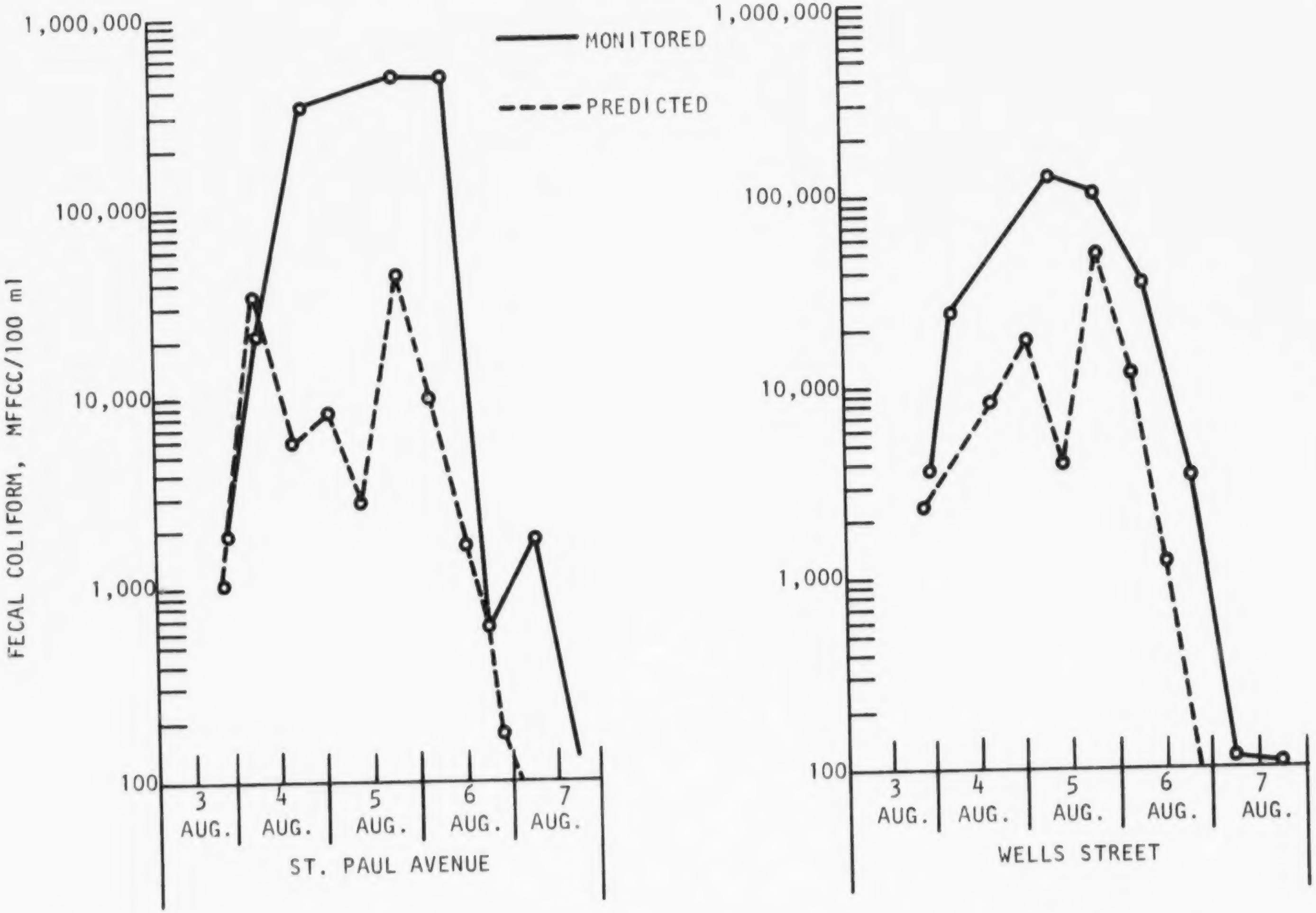


Figure 61. Wet weather fecal coliform calibration results, August 4-7, 1977 (Survey 4).

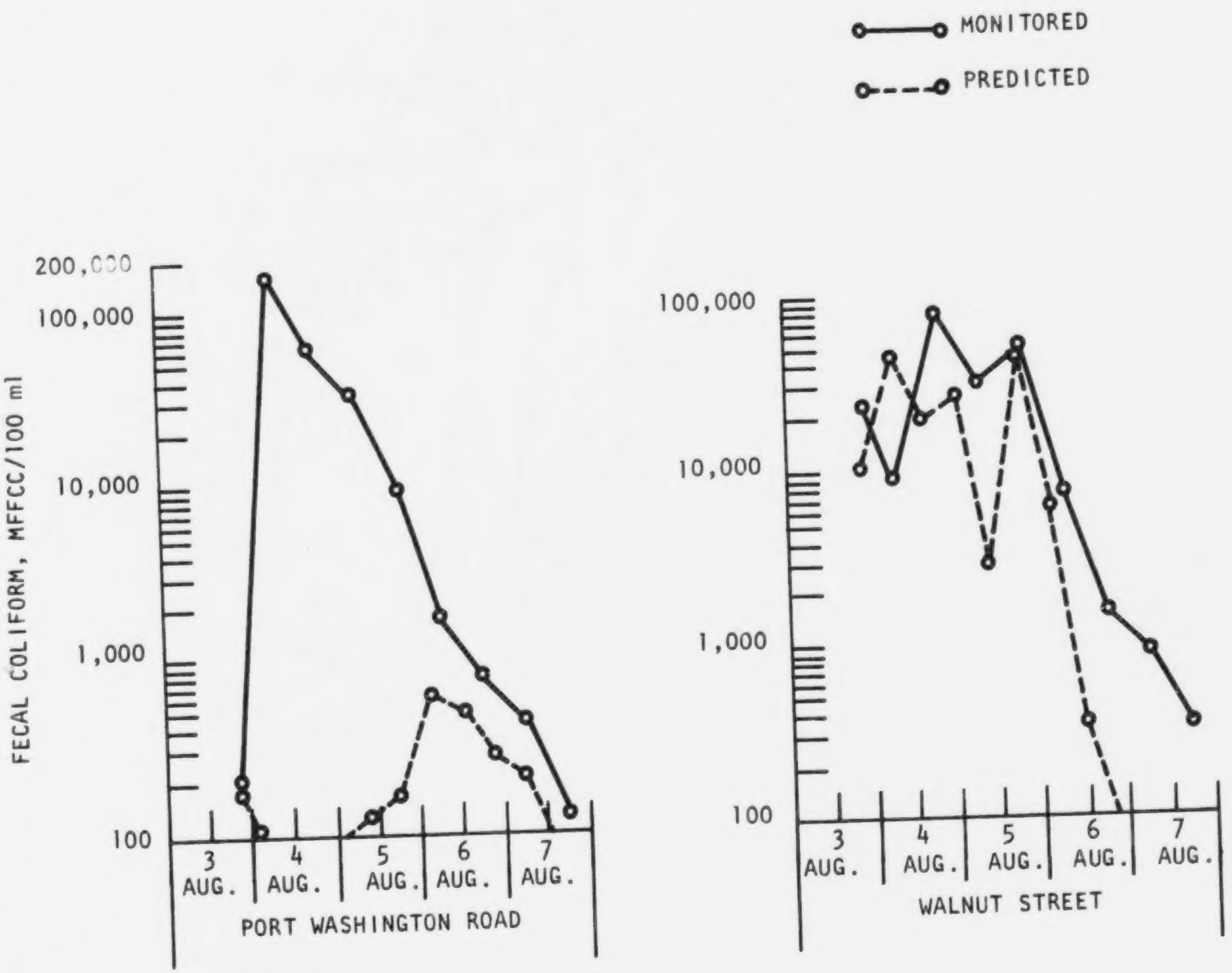


Figure 62. Wet weather fecal coliform calibration results, August 4-7, 1977 (Survey 4).

The predicted peak concentrations of fecal coliforms are reasonably close to observed values. The low upstream load of fecal coliforms has little effect on the predicted concentrations in the lower river due to the large load input from the CSO area. This lack of an upstream load of fecal coliforms may be responsible for a faster predicted decline in fecal coliforms than observed. On the basis of these results, the assumption of a slight die-off rate of fecal coliforms as discussed in the dry weather calibration section, appears to provide a reasonable estimate of fecal coliform trends in the Milwaukee River. Calibration for fecal coliforms is considerably different than calibration for dissolved oxygen for two reasons:

1. The only adjustment factors are the loads and the growth/die-off coefficient.

2. There is such a large amount of variability in fecal coliform results that predicted values within an order of magnitude of the observed values are acceptable.

SECTION 7

EVALUATION OF CSO IMPACT

The contribution of CSO to the DO and fecal coliform levels in the Milwaukee River can now be quantified using the relationships developed in the previous sections of this report. The source of the water quality impacts for all rainfall-runoff events can be deciphered and discussed through the long term operation of the model network that is used to simulate instream water quality conditions. Once the contribution is fully described, the methodologies will be related to other sites that do not contain downstream influences such as Lake Michigan.

DISSOLVED OXYGEN IMPACTS

In order to evaluate the contribution of CSO to the DO impacts within the Milwaukee River, a sensitivity analysis using the instream model network with various CSO loads was conducted. The STORM and instream model were run for one year of rainfall for the period April through October when non-frozen ground and non-ice cover conditions are present. In order to measure the differences in the DO curves that are computed by Harper's model, the technique shown in Figure 63 was utilized. The shaded area below 5.0 mg/l standard represents the magnitude of the dissolved oxygen impact which has the units of mg-day/l. The magnitude measurement represents the difference in water quality between a given standard and the predicted values. Another measurement which is used to quantify the DO impact is the number of days of violation that occur within the simulated time period. Thus, any part of a day which contains a violation of the standard will be listed as such. Figure 63 presents the data where there are eight DO values listed per day in the model output. The 5.0 mg/l standard is violated during day 2 and 3 of the period with day 2 having four values below the 5.0 mg/l standard and day 3 having one. Two days of violation will then be listed along with the magnitude of the violations for this period.

In order to quantify the sensitivity of the river to the loads generated by CSO, an extreme year of rainfall was selected from the 38 years of record available for the Milwaukee area. The predicted flow and quality from STORM for each storm event were input to Harper's model to generate the frequency and magnitude of DO violations at each site for the April through October period using mean monthly low flows. The DO standards for determining the magnitude and frequency of violations were 5.0 mg/l for sites upstream of the North Avenue Dam and 2.0 mg/l for sites downstream of the dam as discussed in Section 3.

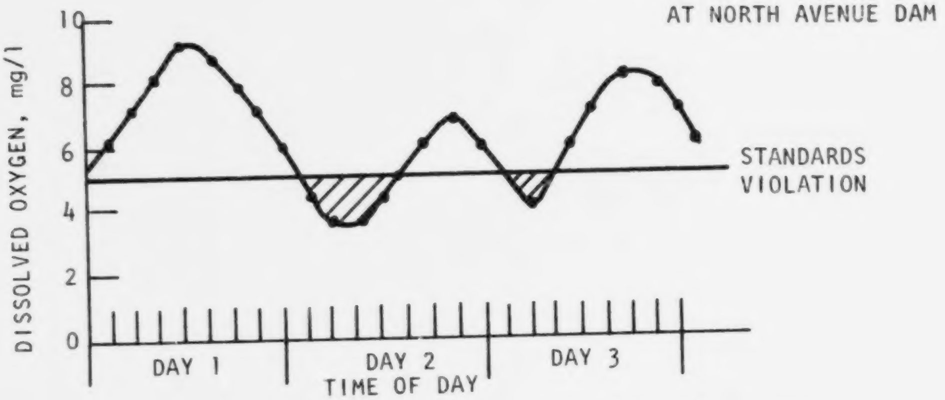


Figure 63. Typical model output for determining frequency and magnitude of dissolved oxygen violations.

Next, the predicted pounds of BOD, suspended solids and numbers of fecal coliforms for each overflow event during the year in the CSO area were doubled and loaded into Harper's model with the same storm sewer and upstream boundary loads as the previous model runs. This technique was used to determine the change in DO quality with this 200 percent increase in CSO pollutant load. Similarly, the CSO load and flow was also completely removed to simulate the complete elimination of CSO. These three simulations were run using the CSO sediment scour potential for each runoff event and they were run without the scour mechanism in order to quantify the differences between these conditions. The results of these model runs are listed in Table 26 for three sites within the study area. A graphical representation of the differences in these simulations is shown in Figure 64 for both the with and without scour mechanism.

TABLE 26. THE DISSOLVED OXYGEN IMPACT OF CSO LOADS VARIATIONS USING AN EXTREME RUNOFF YEAR

	North Avenue			Walnut Street			St. Paul Avenue		
	Existing	200%	Zero	Existing	200%	Zero	Existing	200%	Zero
DO impact magnitude, mg-days/l									
with scour	43.8	52.7	34.6	17.2	30.7	0	135.5	165.1	5.0
w/o scour	-	-	-	5.3	18.1	0	12.3	47.3	0
Days DO violations									
with scour	32.0	33.0	31.0	18.0	33.0	0	110.0	117.0	29.0
w/o scour	-	-	-	9.0	16.0	0	19.0	43.0	0

The data of Table 26 and Figure 64 provide a means of evaluating the contribution of CSO to the impacts on DO at various locations within the Milwaukee

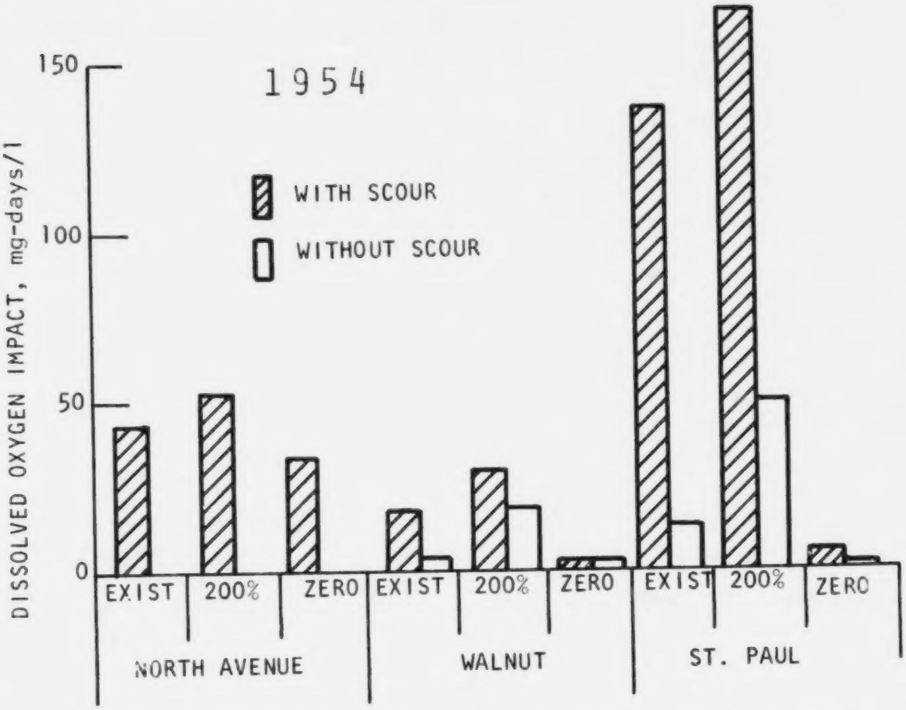


Figure 64. Sensitivity of dissolved oxygen results in CSO loads.

River. For example, the North Avenue data shows that the doubling of the CSO load results in an increase of approximately 20 percent in the magnitude of the DO violations of the 5.0 mg/l standard. Removal of all CSO at this site results in a 21 percent decrease in the magnitude. Sediment scouring from submerged CSO outfalls does not occur in the portions of the river upstream of the North Avenue Dam. Therefore, the DO impacts at North Avenue are strictly a function of the wet weather loads and the low flow conditions that were used for this analysis. At Walnut Street, where sediment scouring does occur, the differences are more pronounced. Doubling the CSO load results in a 78 percent increase in the magnitude of the standard violations of 2.0 mg/l. Complete removal of the CSO load results in zero magnitude below the standard. The contribution of sediment scour to these violations, as determined using the without scour results, is approximately 69 percent. The existing magnitude of DO violations is 17.2 mg-days/l with scour. This decreases to 5.3 mg-days/l (69 percent decrease) when the scour mechanism is removed. At higher CSO loads (200 percent), the sediment contribution to the magnitude of DO violations is reduced to approximately 37 percent at Walnut Street. This reduction in influence of sediments is due to the higher dissolved and suspended loads utilized in the 200 percent CSO load simulation.

The St. Paul Avenue site has the most drastic changes in water quality when comparing the sensitivity of various loads. By doubling the CSO load, the DO magnitude below 2.0 mg/l increases by 22 percent while removing the CSO load reduces the impact by 96 percent. Removal of the scour mechanism from the model network reduces the impact of the existing CSO load by 91 percent when compared to the with scour alternative. This result indicates the significance of the scouring of sediments on DO conditions in the lower portions of the Milwaukee River. In comparison, the CSO loads have only a minor influence on DO levels during the low flow conditions used for this simulation.

The sensitivity of the DO conditions in the Milwaukee River to the river flow was evaluated using the mean monthly flows rather than the mean low flows as in the analyses previously presented. The results of these simulations are shown in Table 27.

TABLE 27. RESULTS FOR INSTREAM FLOW CONDITIONS USING AN EXTREME RUNOFF YEAR

	North Avenue		Walnut Street		St. Paul Avenue	
	Low flows	Average flows	Low flows	Average flows	Low flows	Average flows
DO impact magnitude, mg-days/l						
with scour	43.8	0	17.2	0	135.5	24.3
w/o scour	-	-	5.3	0	12.3	0
Days DO violations						
with scour	32.0	0	18.0	0	110.0	35.0
w/o scour	-	-	9.0	-	19.0	0

Changes in the DO impact when instream flows are increased to average values is as dramatic as the removal of the scour mechanism. For example, the St. Paul Avenue site has a decrease in the magnitude of DO violations of approximately 82 percent when the river flow changes from low flow to mean flow conditions using the existing CSO loads. The ability of the river to flush out the low DO conditions and replace them with oxygenated upstream flow is evident from this analysis.

The previous simulations of water quality are a result of the predictions using the instream water quality model. In order to check the validity of the predictions of the model, the monitoring data generated in the intensive surveys was used to find similar cases which would tend to back up these findings. Figures 65 and 66 present the monitored and predicted DO levels at St. Paul Avenue during Survey 4 and 5, respectively. As was discussed in Section 4, survey 4 occurred during low flow conditions with over 1.0 in. (2.5 cm) of rainfall on portions of the CSO area. The dotted line used in this figure represents the predicted DO level without the sediment scour mechanism. The difference between these curves is what would

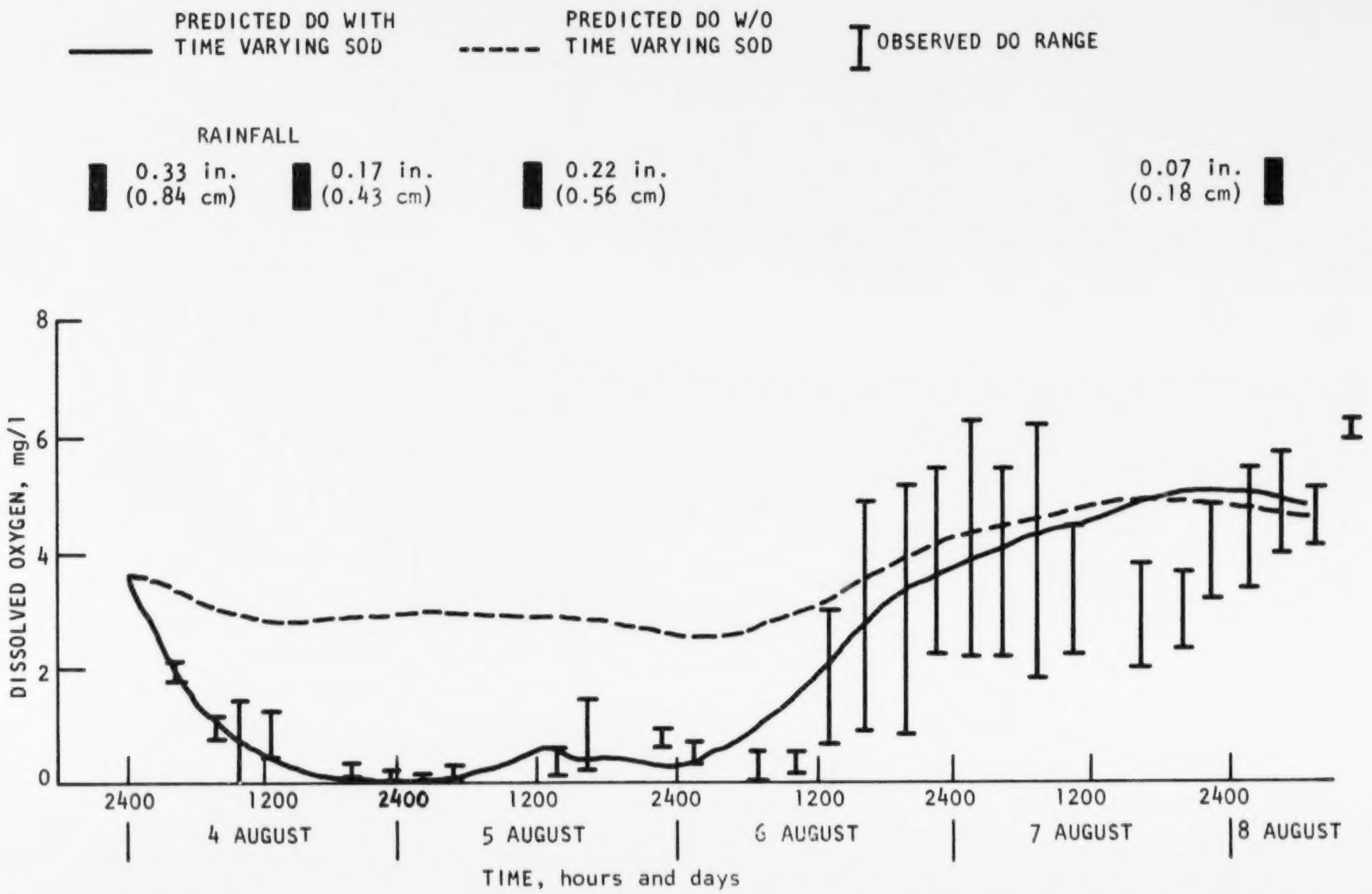


Figure 65. Comparison of the instream model results for with and without the time varying SOD (scour) for St. Paul Avenue, August 3-8, 1977 (Survey 4).

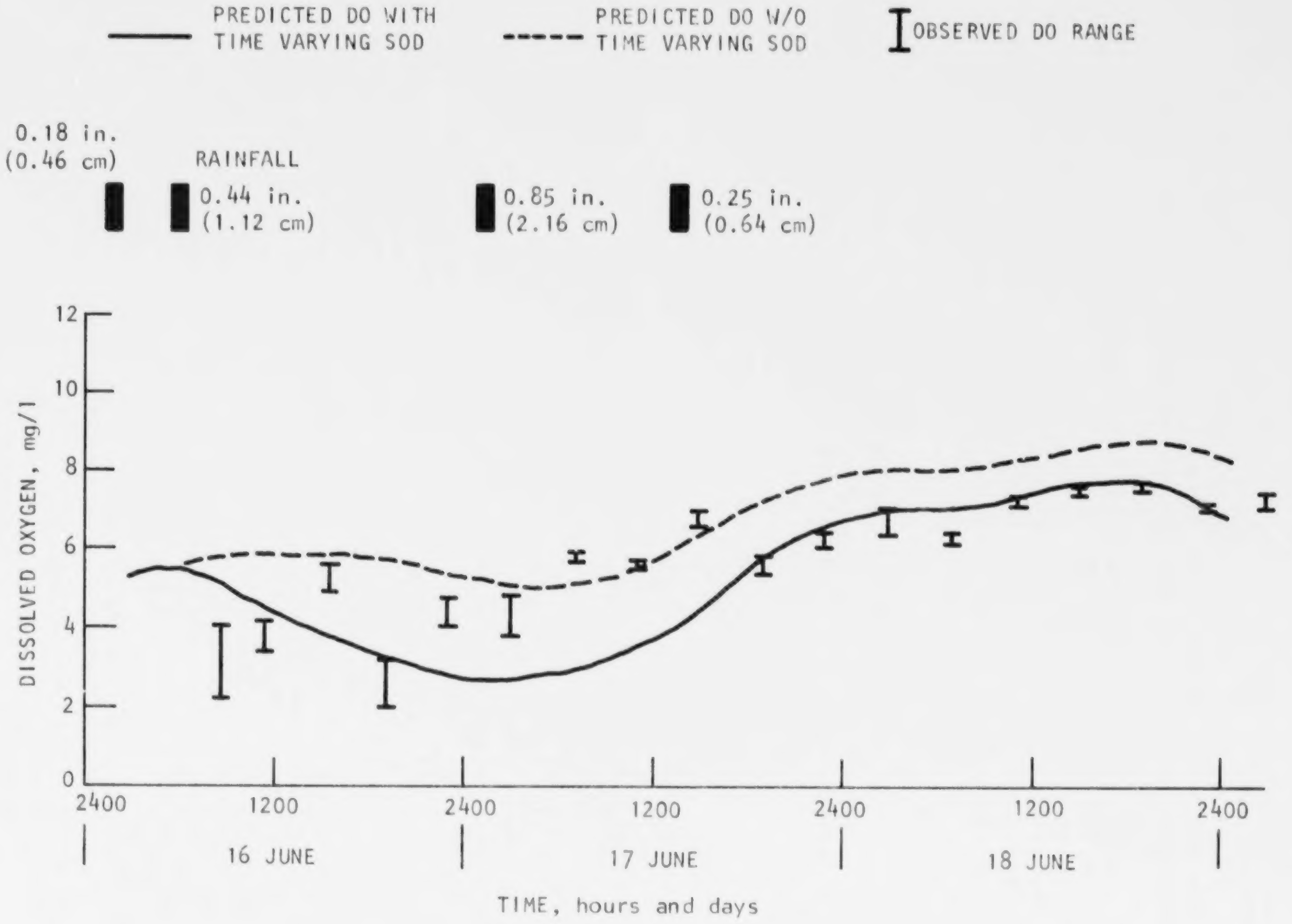


Figure 66. Comparison of instream model results for with and without the time varying SOD (scour) for St. Paul Avenue, July 16-18, 1978 (Survey 5).

be expected when the scour mechanism is removed during these flow conditions. Figure 66 represents the DO data from Survey 5 which occurred during high flows following approximately 2.0 in. (5.1 cm) of rainfall. Note that the DO curves representing the with and without scour cases are closer together and that the depressed DO levels shown in Figure 65 do not occur in this event. The effects of sediments are masked by high flows that remove the deoxygenated waters to the lake and replace them by upstream flows.

Figures 67 and 68 represent the monitored and computed DO levels for the Walnut Street site during Surveys 2 and 5, respectively. Survey 2 was conducted during low flow conditions while Survey 5 occurred during high flows. The differences in the with and without scour computed curves is most noticeable during the low flow conditions of Survey 2. The monitored DO levels remain reasonably high for this site throughout both surveys because of the reduced influence of sediments when compared to sites that are further downstream.

The monitoring and modeling results were illustrated in the four previous figures to verify the sensitivity of the river to the different flow rates that has been developed in the preceeding sections of this report. Obviously, the monitoring program could not collect data that represents the non-scour simulation since this is physically impossible. But the sediment studies and the verification of the instream model have provided an accurate means of evaluating the CSO contribution to the DO impact in the river.

FECAL COLIFORM IMPACTS

The contributions of CSO discharges to the fecal coliform levels in the Milwaukee River were determined using the same techniques as with the DO data. Thus, the sensitivity to CSO loads and the effect of river flows on the number of days of standards violations were investigated to produce an estimate of the source of high coliform levels observed in the monitoring program. A major problem in deciphering the CSO contribution revolved around the loads generated from the cross-connected storm sewers in the upper reaches of the river. A decision was made in the Milwaukee CSO project (11) to ignore these sanitary overflows because of an ongoing program of the Milwaukee Metropolitan Sewerage District to remove these cross-connections in the near future. It was decided to evaluate the fecal coliform impacts under this same assumption in the subject project. This means that the instream model was loaded with the CSO fecal coliform load as predicted by the calibrated STORM model while the upstream contribution of fecal coliforms was estimated as though the storm sewers in these areas were not cross-connected. The differences between the instream model predictions and the monitoring data from the intensive surveys can then be attributed to the cross-connected storm sewers.

Table 28 lists the fecal coliform results of the instream model for variations in CSO loads for the extreme runoff year during average and low flow conditions. The number of days of standards violations takes into account the differences in standards between the upstream and downstream

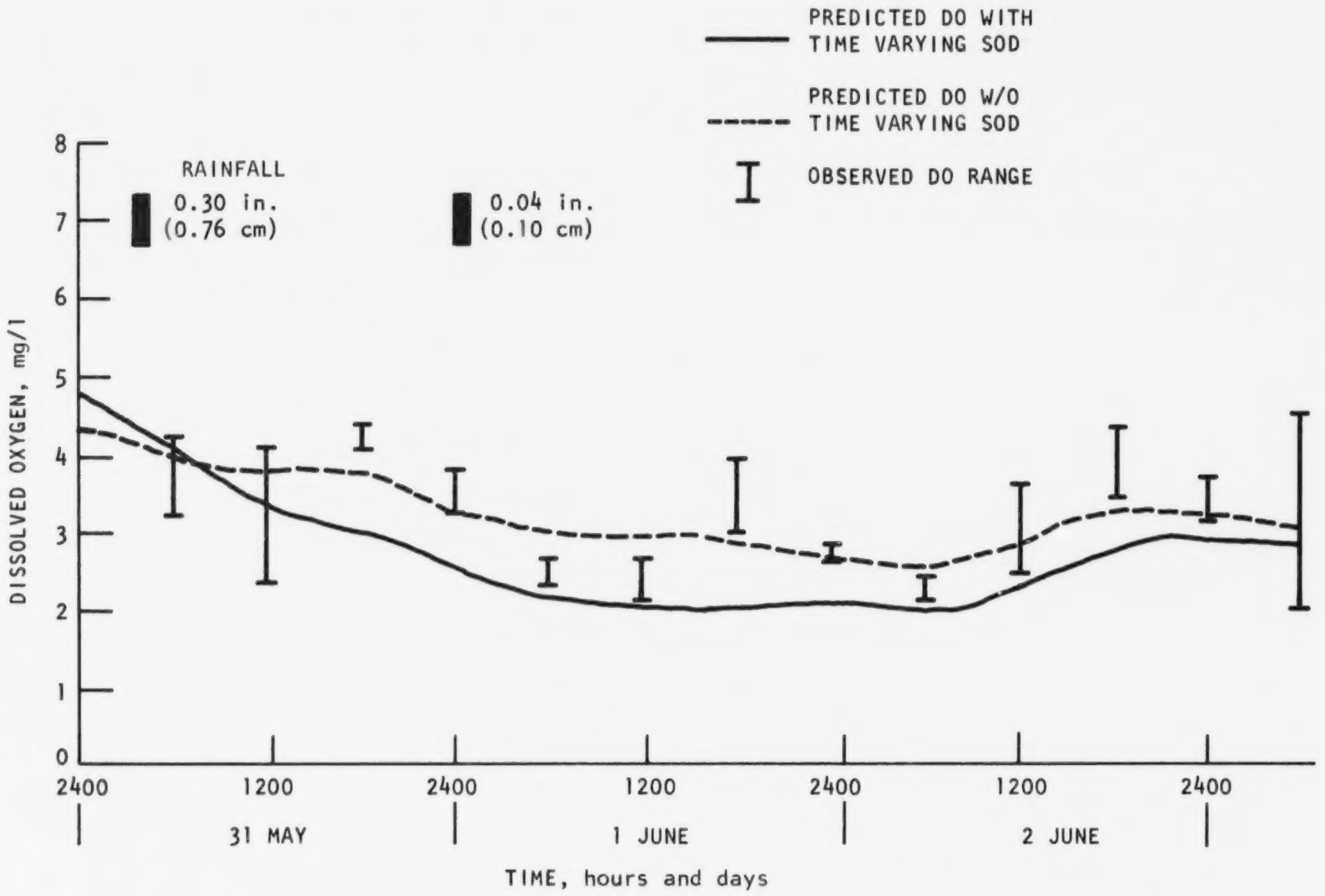


Figure 67. Comparison of the instream model results for with and without the time varying SOD (scour) for Walnut Street, May 31-June 3, 1977 (Survey 2).

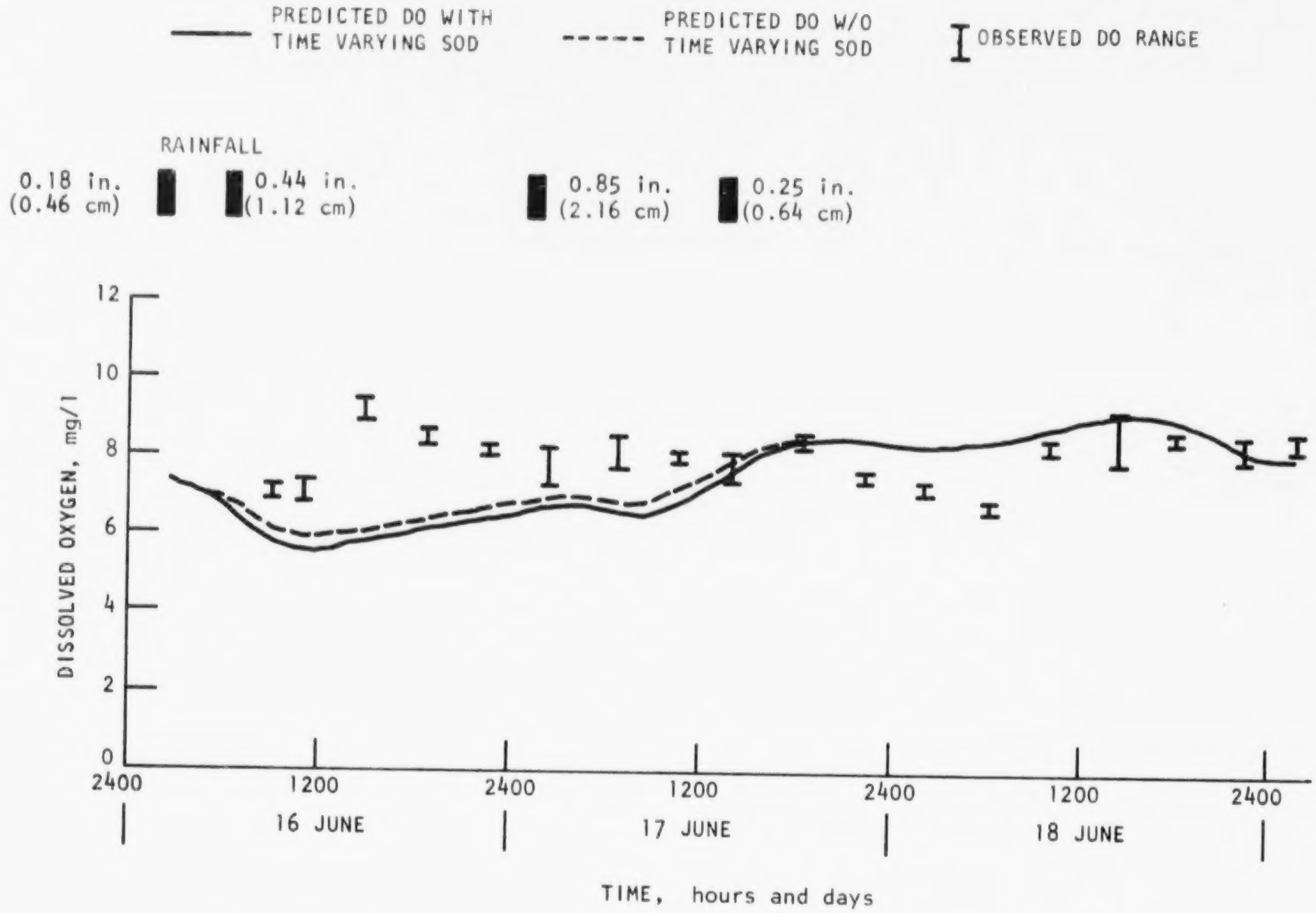


Figure 68. Comparison of instream model results for with and without the time varying SOD (scour) for Walnut Street, June 16-18, 1978 (Survey 5).

reaches. Thus, 40 days of violation at the North Avenue site is in reference to the 200/100 ml standard while the lower reaches are in violation of the 1000/100 ml standard. The most significant result of these simulations is the absence of standards violations when the entire CSO load is removed from the the model network. Another point of interest is that doubling of the CSO load results in an approximate 50 percent increase in the number of days of violation during low flow conditions. The findings also indicate that there are more days of violations at Walnut Street than at North Avenue despite the difference in the standards. This supports the results of the intensive monitoring surveys which showed increasing levels of fecal coliforms as the distance downstream increased. This reflects a greater proportion of the CSO area contributing to the river. The major source of the fecal coliform levels in the river is thus, the CSO area. The St. Paul Avenue site has less days of violation than the Walnut Street site because of dilution from Lake Michigan.

TABLE 28. DAYS OF FECAL COLIFORM VIOLATIONS FOR VARIATIONS IN CSO LOAD AND RIVER FLOW CONDITIONS USING AN EXTREME RUNOFF YEAR

	North Avenue			Walnut Street			St. Paul Avenue		
	Existing	200%	Zero	Existing	200%	Zero	Existing	200%	Zero
Average flow	28	NA[1]	0	31	NA[1]	0	30	NA	0
Low flow	40	57	0	40	61	0	33	50	0

[1] Not available.

The total numbers of fecal coliforms discharged to the river and arriving at the upstream boundaries can be calculated for the year of analysis. Non cross-connected storm sewers were assumed for these calculations. The results are shown in Table 29.

TABLE 29. YEARLY FECAL COLIFORM LOADS

Source	counts
Upstream (arriving)	3.1×10^{13}
Storm sewers	5.3×10^{14}
CSO	2.8×10^{17}

These data show that the relative magnitude of the CSO contribution is much greater than the sum of the other two sources.

APPLICATION TO OTHER STUDY AREAS

The results of this study are site specific because of the unique nature of the receiving waters. In order to relate the findings of this effort to other study areas, the specific results will be reduced to a more general consideration. The most significant aspect of the subject project is the relationship of the bottom sediments in the river to the DO levels observed during the monitoring program. Since the source of the sediments is partially related to the CSO area (approximately 40 to 50 percent of the loading of oxygen demanding materials to the sediments are attributable to CSO), the possibility in other study areas for similar buildup of bottom materials must be evaluated in the design of a monitoring program. Thus, larger urban areas than the Milwaukee area may have a greater percentage of the drainage area served by urban storm sewers. These discharges may produce enough settleable material to cause the build-up of oxygen demanding sediments in those portions of the receiving waters where scouring velocities are not present during low flow conditions. These discharges coupled with combined sewer overflows could produce a sediment oxygen demand which would have a serious effect on the DO conditions of the river during wet weather.

The contribution of CSO to the DO impacts in the Milwaukee River has been quantified as being partially due to the dissolved and suspended loads and significantly due to the scour of sediments by submerged CSO outfalls. The most significant contribution is the scouring potential of the submerged outfalls. This mechanism accounts for zero percent of the impacts at the North Avenue site and 91 percent at the St. Paul Avenue site. The lower reaches of the river represent a deep, slow moving portion of the river while the upstream site is more shallow and represents a free flowing stream. The Walnut Street site has conditions which range between these two extremes since the river is not as deep nor is the influence of the lake as pronounced as at sites further downstream. The contribution of the scour mechanism to the DO impacts at this site ranges from 37 to 69 percent. The remainder of the impact is due to the CSO loads. The fecal coliform impacts at all sites are solely attributable to the CSO loads.

The removal of submerged outfalls within the CSO drainage area of the Milwaukee River was simulated by removing the scour mechanism from the receiving water model. The change in DO impacts after the removal of this mechanism was significant. In other receiving waters, the existance of submerged outfalls may have effects on the DO levels of the stream which should be investigated in the monitoring program before complex models are applied to the stream to predict the DO concentrations. The scour and dispersion of the CSO discharges may have dramatic effects on the quality of the receiving stream which are normally not included in most analyses. Investigations of the source and mechanism of water quality impacts in receiving waters should consider the scouring potential of CSO discharges and the dispersion of CSO during the sampling efforts.

SECTION 8

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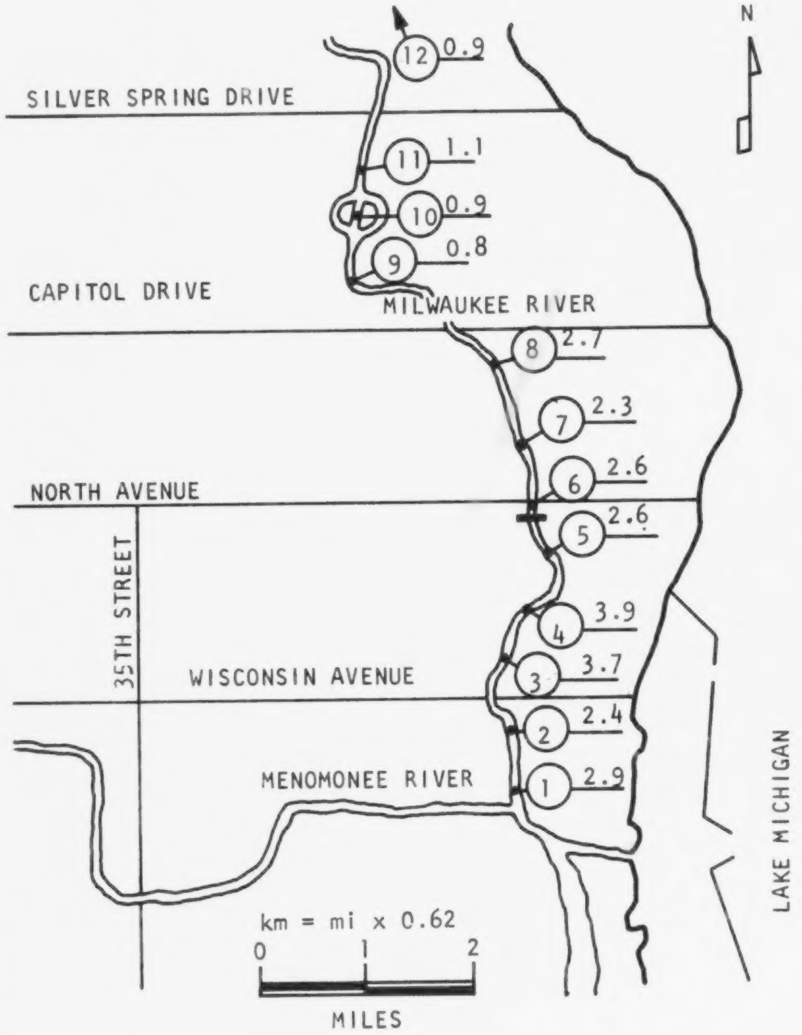
pp. 157-181, 1962.

APPENDIX FIGURES
(A-1 through A-15)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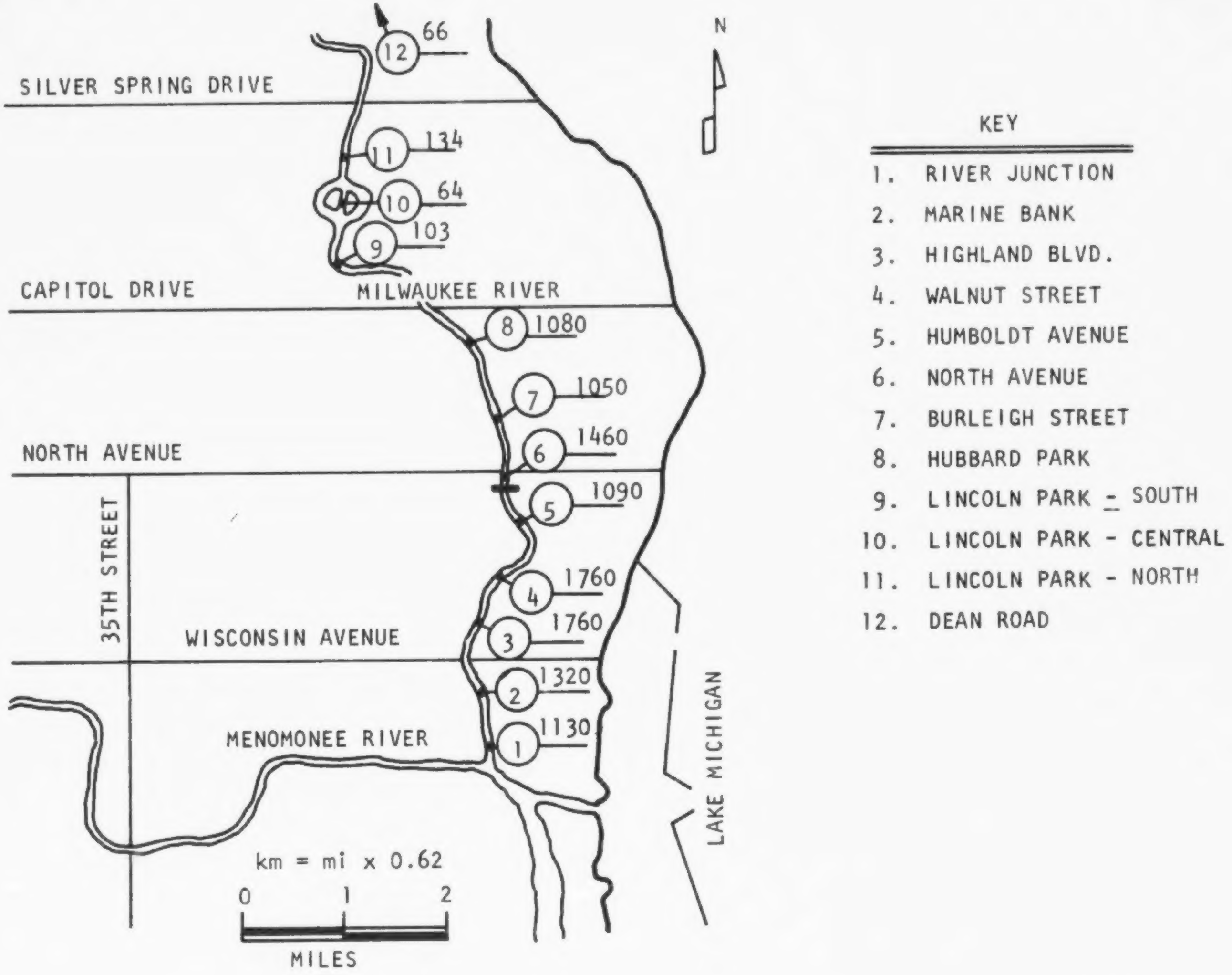
KEY

1. RIVER JUNCTION
2. MARINE BANK
3. HIGHLAND BLVD.
4. WALNUT STREET
5. HUMBOLDT AVENUE
6. NORTH AVENUE
7. BURLEIGH STREET
8. HUBBARD PARK
9. LINCOLN PARK - SOUTH
10. LINCOLN PARK - CENTRAL
11. LINCOLN PARK - NORTH
12. DEAN ROAD

Figure A-1. Total volatile solids (% of sample weight) values in the sediments of the Milwaukee River.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1080
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NORTH AVENUE
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MENOMONEE RIVER
LAKE MICHIGAN
km = mi x 0.62
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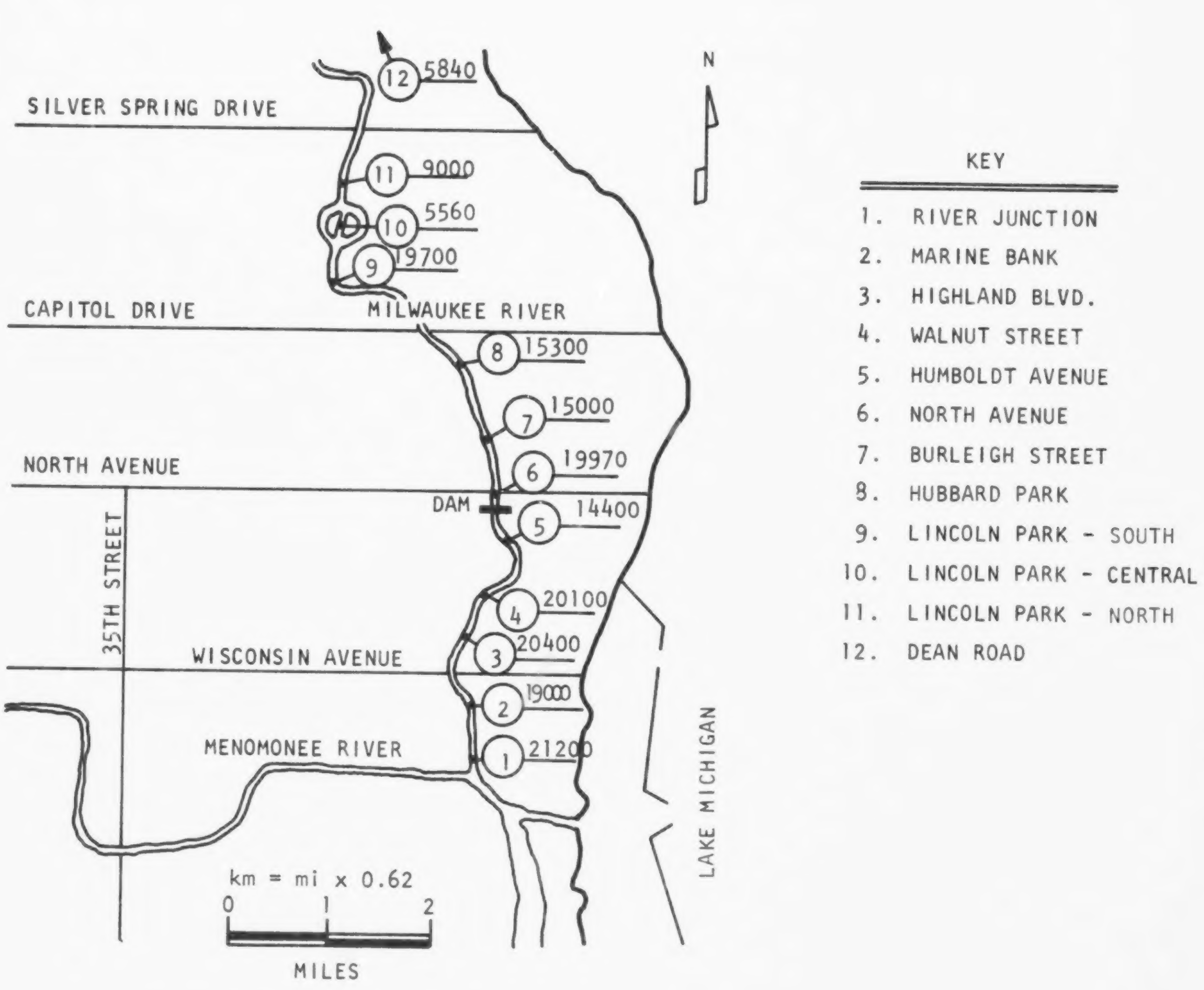
Figure A-2. Phosphorus (mg/kg) values in the sediments of the Milwaukee River.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7 15000
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DAM
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35TH STREET
4 20100
3 20400
WISCONSIN AVENUE
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MENOMONEE RIVER
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LAKE MICHIGAN
km = mi x 0.62
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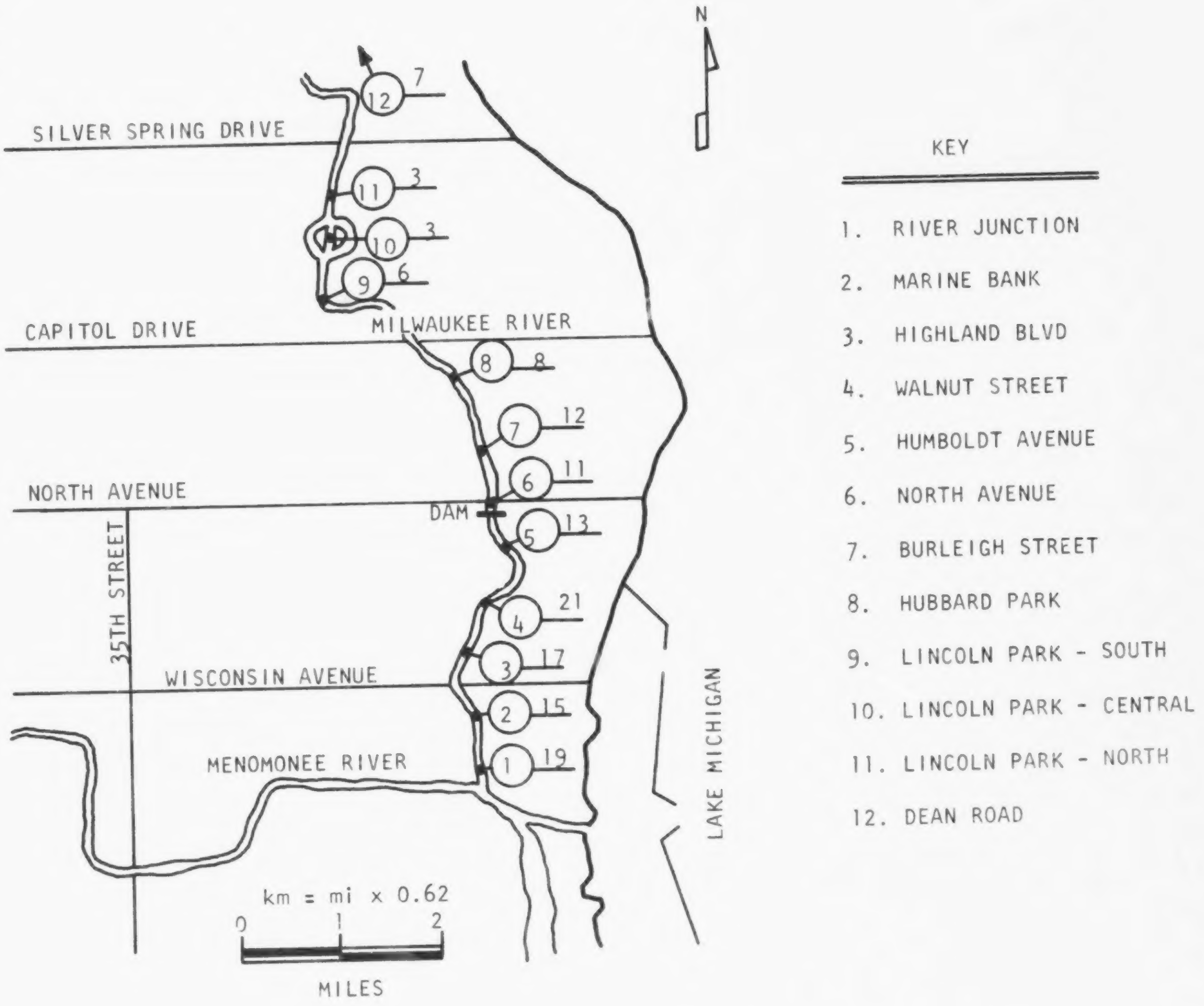
7. BURLEIGH STREET
8. HUBBARD PARK
9. LINCOLN PARK - SOUTH
10. LINCOLN PARK - CENTRAL
11. LINCOLN PARK - NORTH
12. DEAN ROAD

Figure A-3. Iron (mg/kg) values in the sediments of the Milwaukee River.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MENOMONEE RIVER
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km = mi x 0.62
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3. HIGHLAND BLVD
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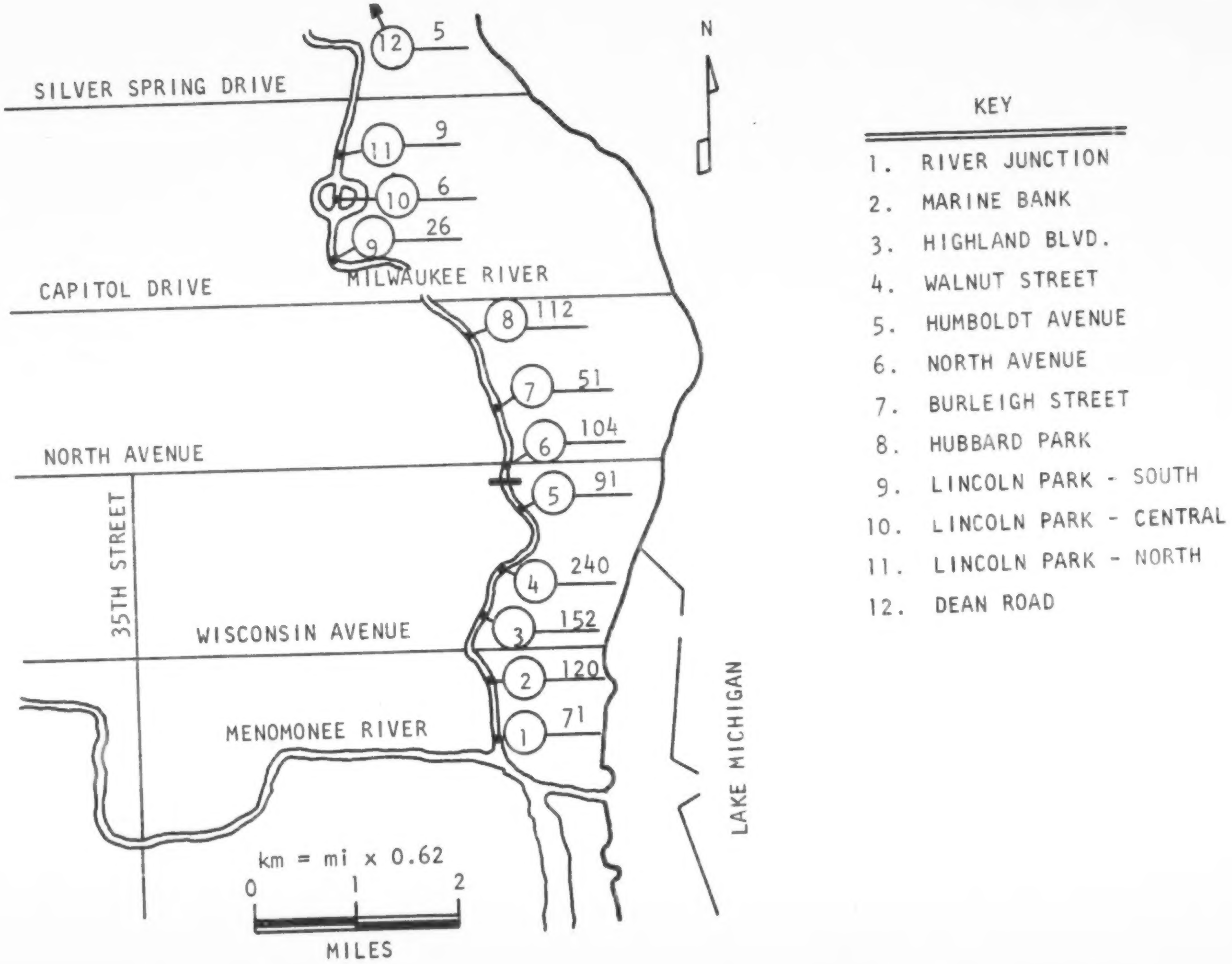
7. BURLEIGH STREET
8. HUBBARD PARK
9. LINCOLN PARK - SOUTH
10. LINCOLN PARK - CENTRAL
11. LINCOLN PARK - NORTH
12. DEAN ROAD

Figure A-4. Cadmium (mg/kg) values in the sediments of the Milwaukee River.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SILVER SPRING DRIVE
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CAPITOL DRIVE
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8 112
7 51
6 104
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5 91
35TH STREET
4 240
3 152
WISCONSIN AVENUE
2 120
1 71
MENOMONEE RIVER
LAKE MICHIGAN
km = mi x 0.62
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3. HIGHLAND BLVD.
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7. BURLEIGH STREET
8. HUBBARD PARK
9. LINCOLN PARK - SOUTH
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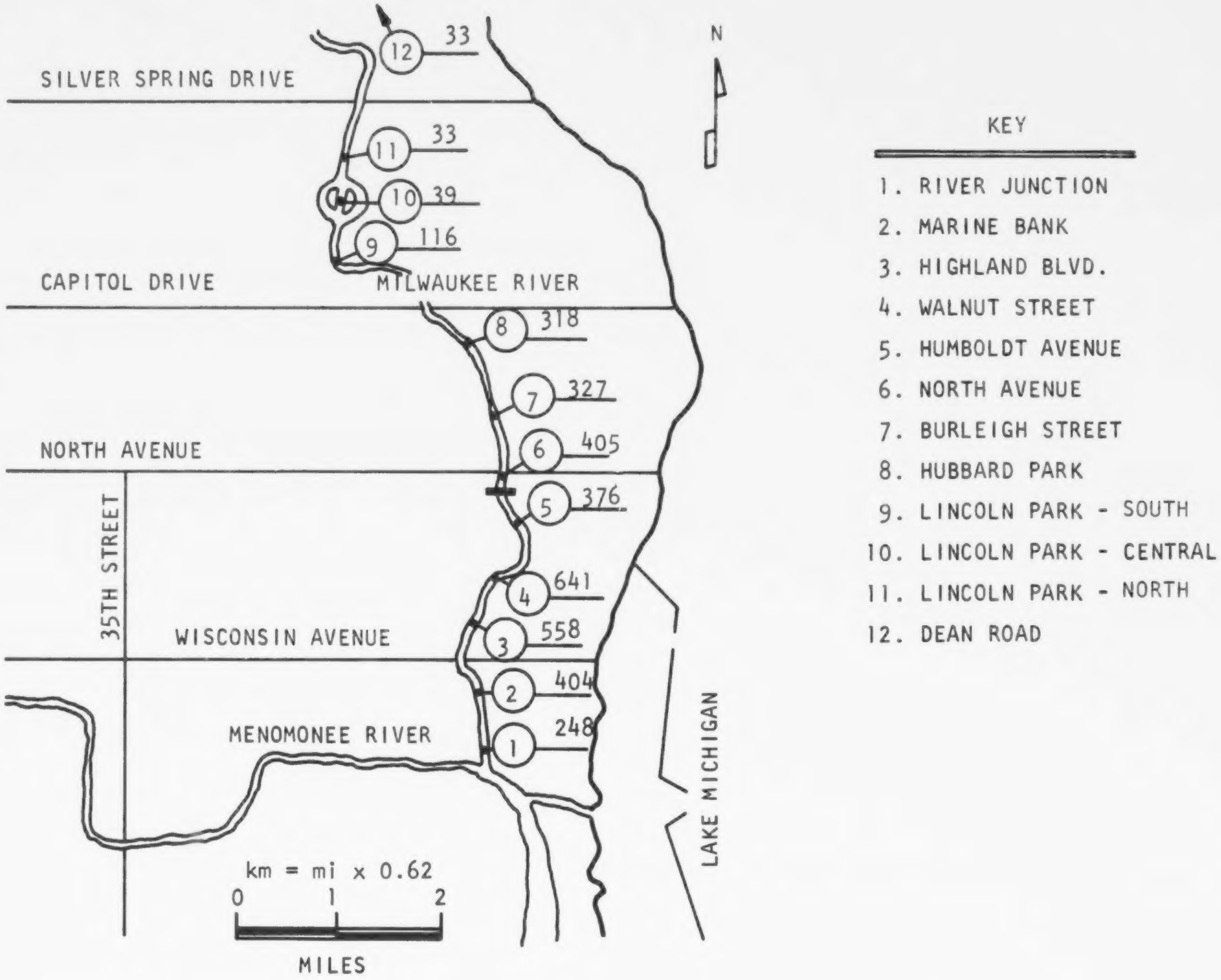
11. LINCOLN PARK - NORTH
12. DEAN ROAD

Figure A-5. Copper (mg/kg) values in the sediments of the Milwaukee River.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NORTH AVENUE
35TH STREET
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MENOMONEE RIVER
LAKE MICHIGAN
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405
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376
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km = mi x 0.62
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Figure A-6. Zinc (mg/kg) values in the sediments of the Milwaukee River.

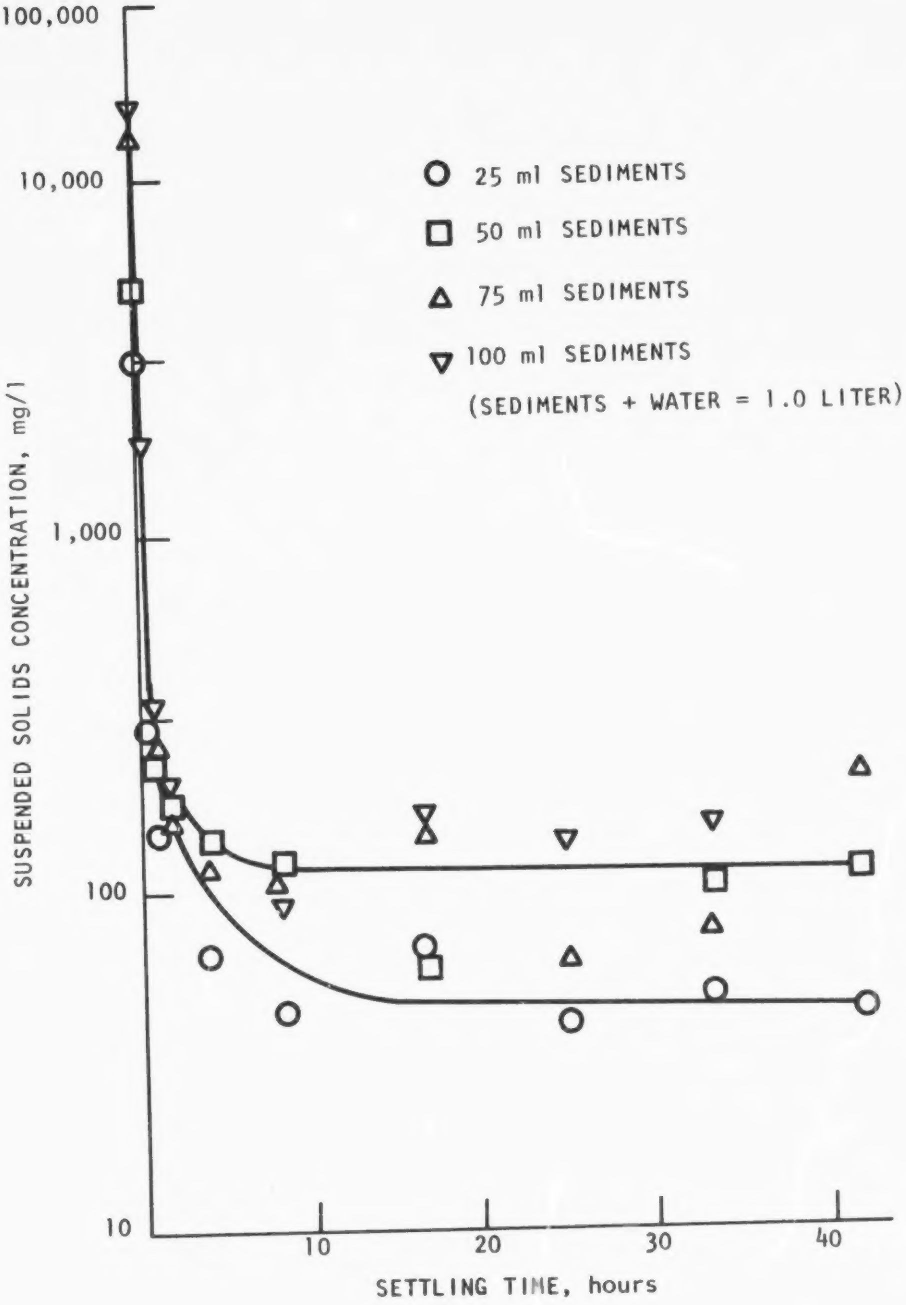

Figure A-7. Settling of Milwaukee River sediments in river water with the samples collected at Wisconsin Avenue in January, 1977.

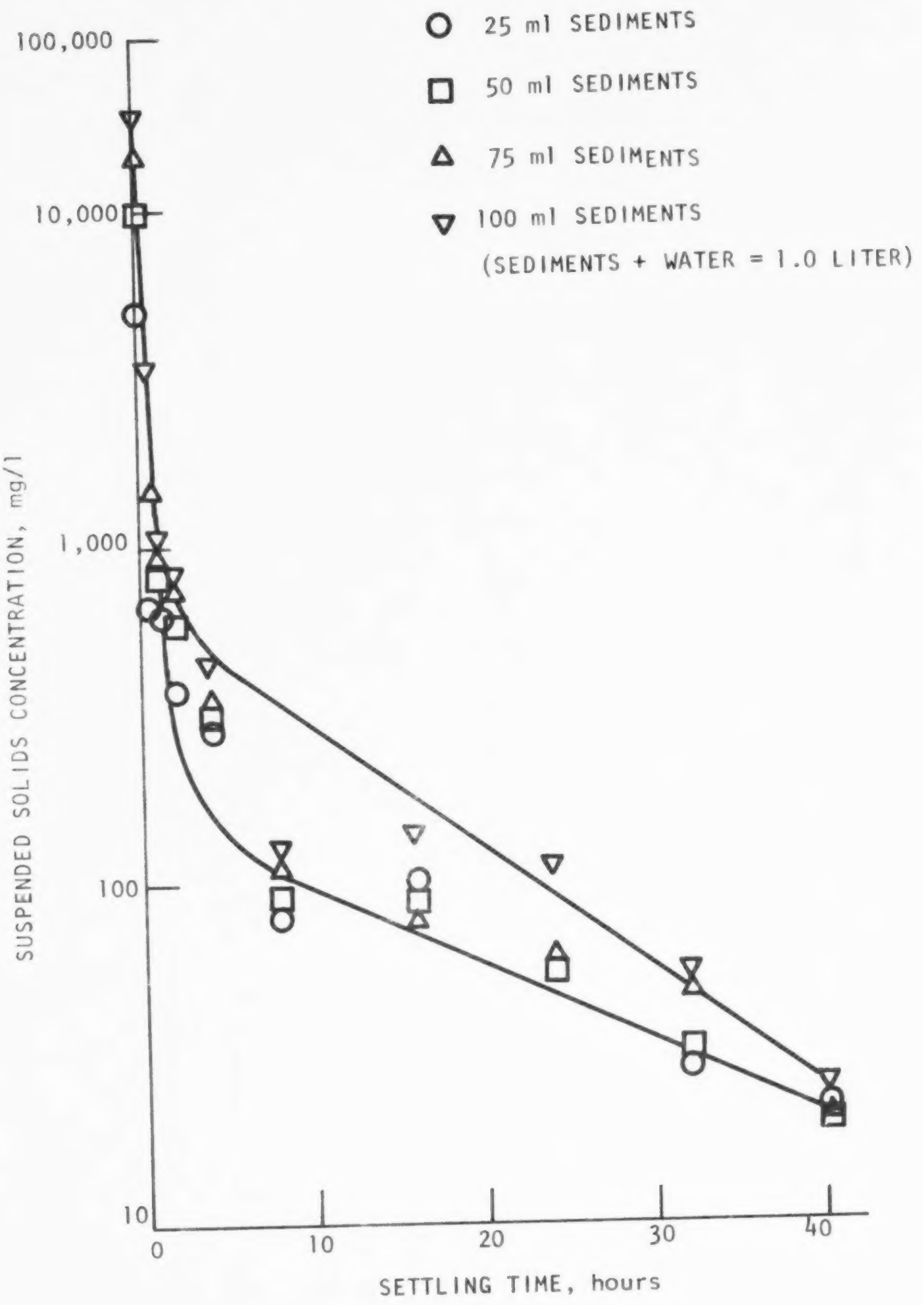

Figure A-8. Settling of Milwaukee River sediments in river water with the samples collected at the junction of the Menomonee and Milwaukee Rivers in February, 1978.

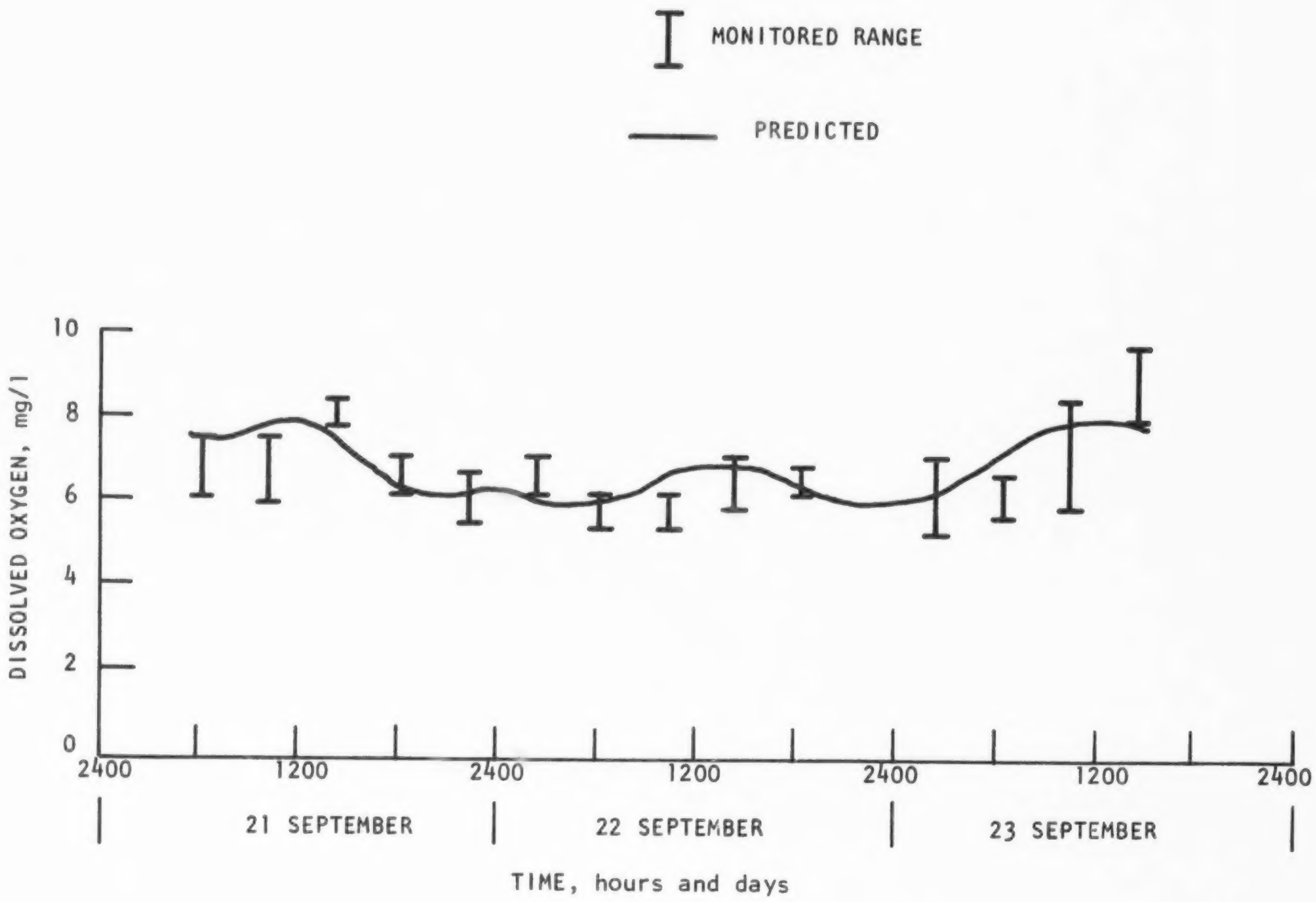

Figure A-9. Dry weather verification results for Capitol Drive, September 21-23, 1976 (Survey 1).

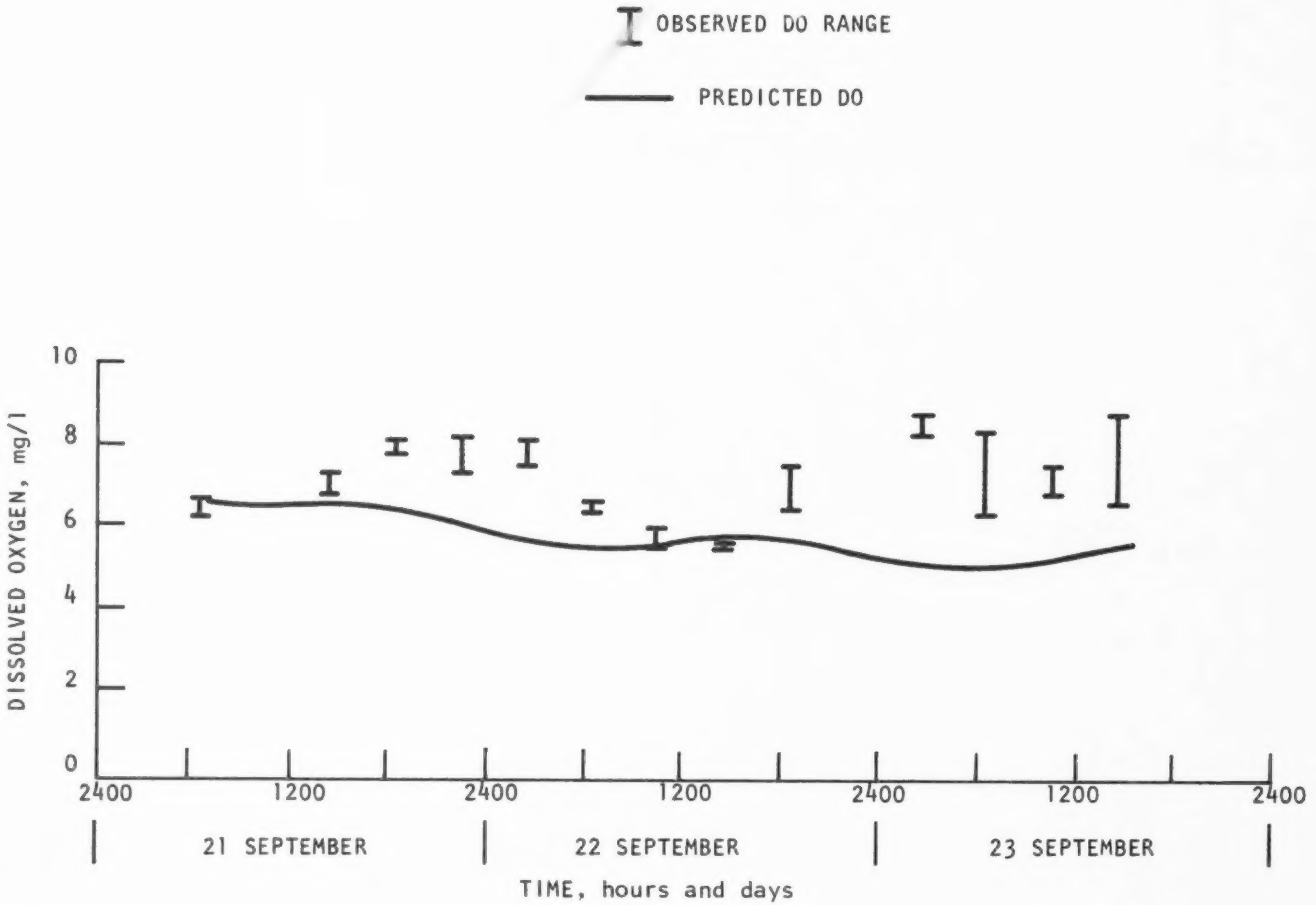

Figure A-10. Dry weather verification results for North Avenue, September 21-23, 1976 (Survey 1).

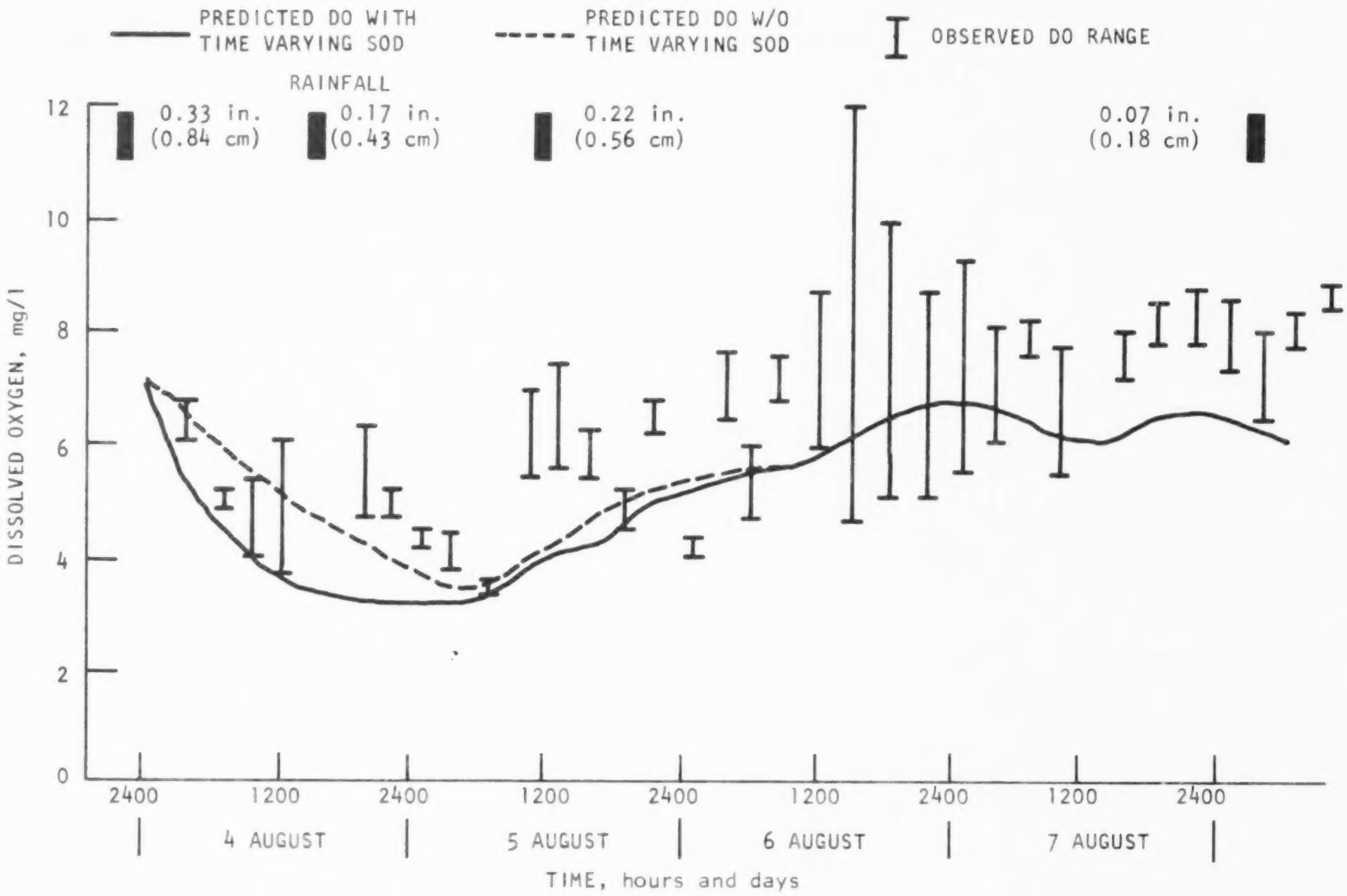

Figure A-11. Wet weather verification results for Walnut Street, August 4-8, 1977 (Survey 4).

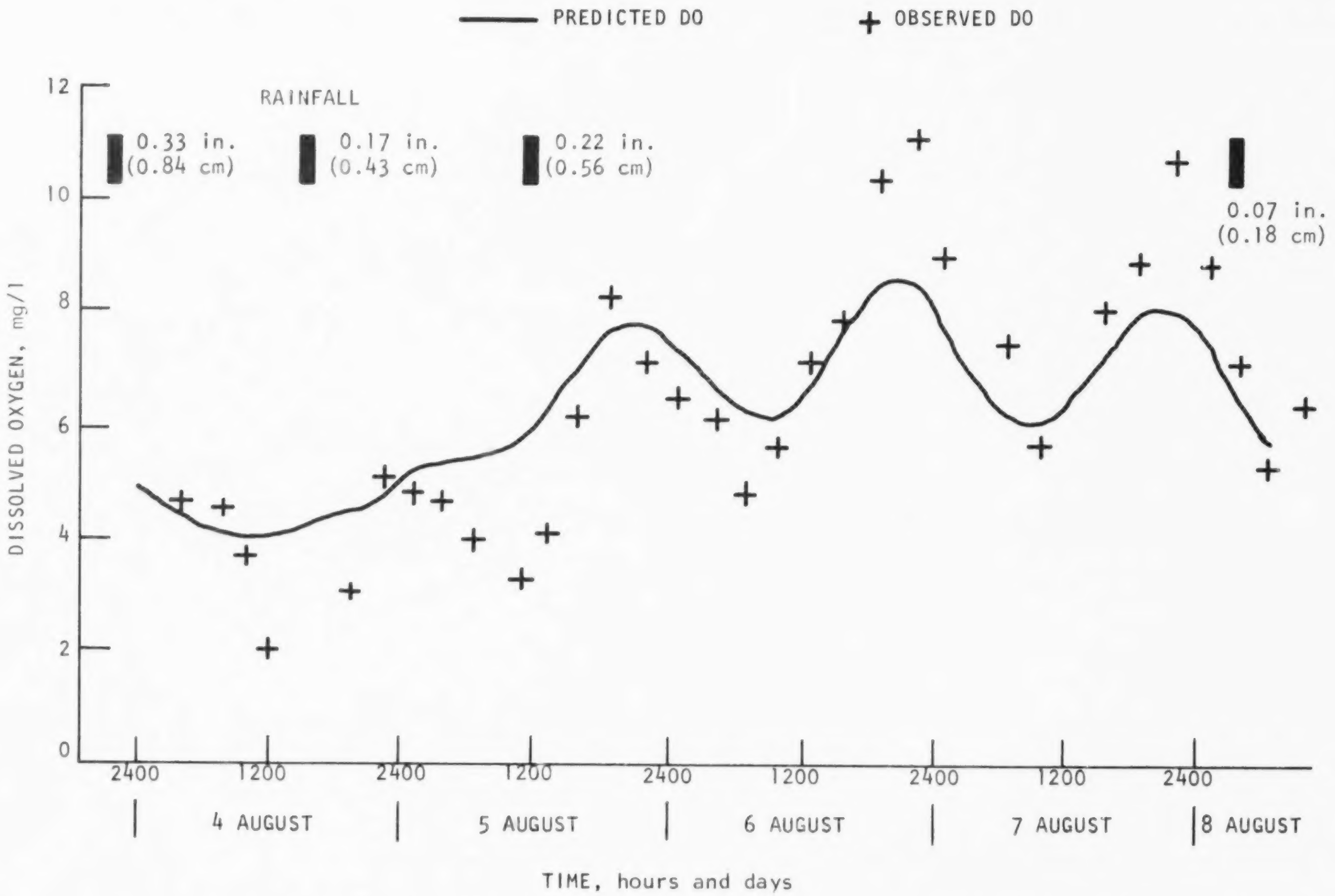

Figure A-12. Wet weather verification results for North Avenue, August 4-8, 1977 (Survey 4).

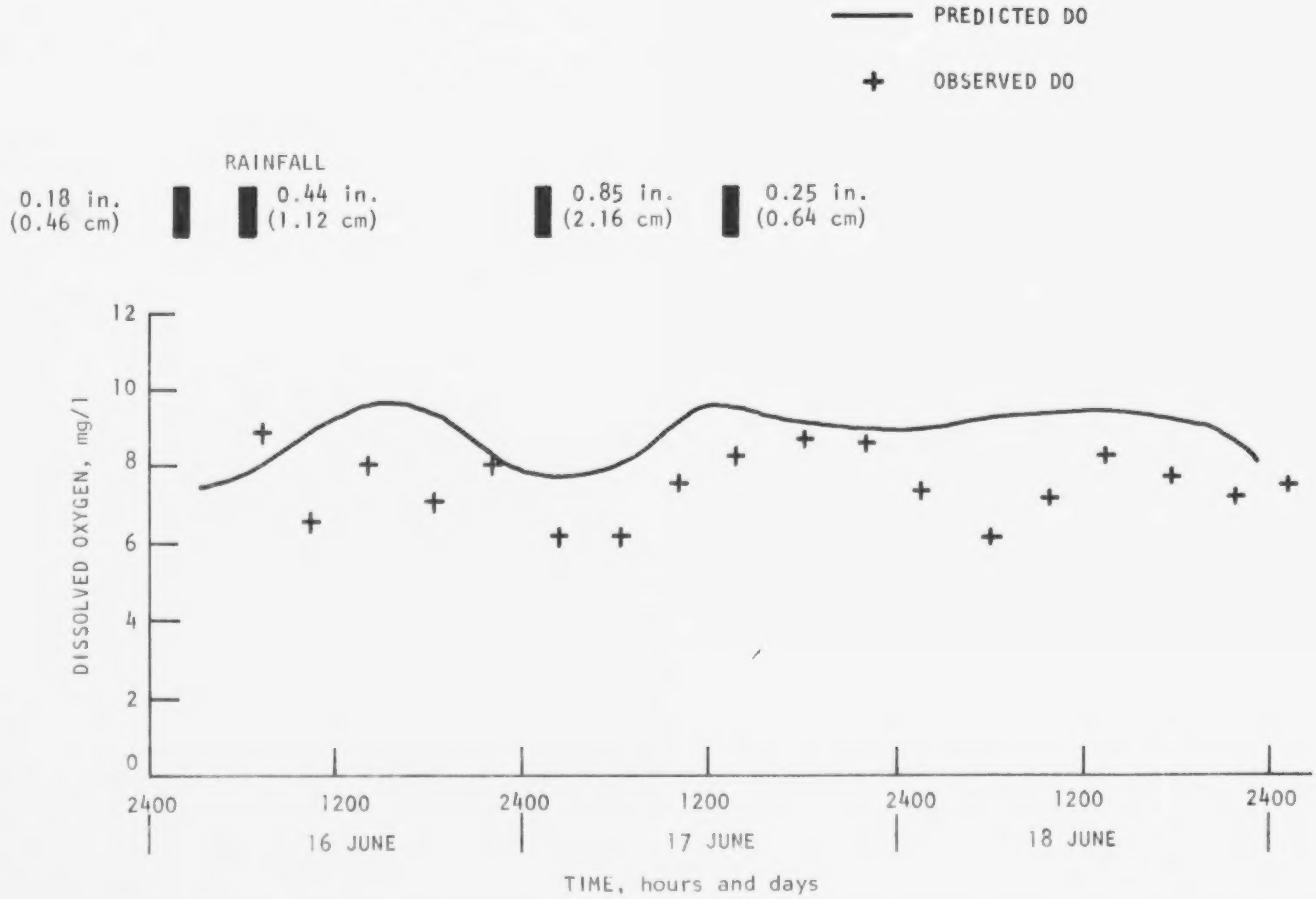

Figure A-13. Wet weather verification results for Port Washington Road, June 16-18, 1978 (Survey 5).

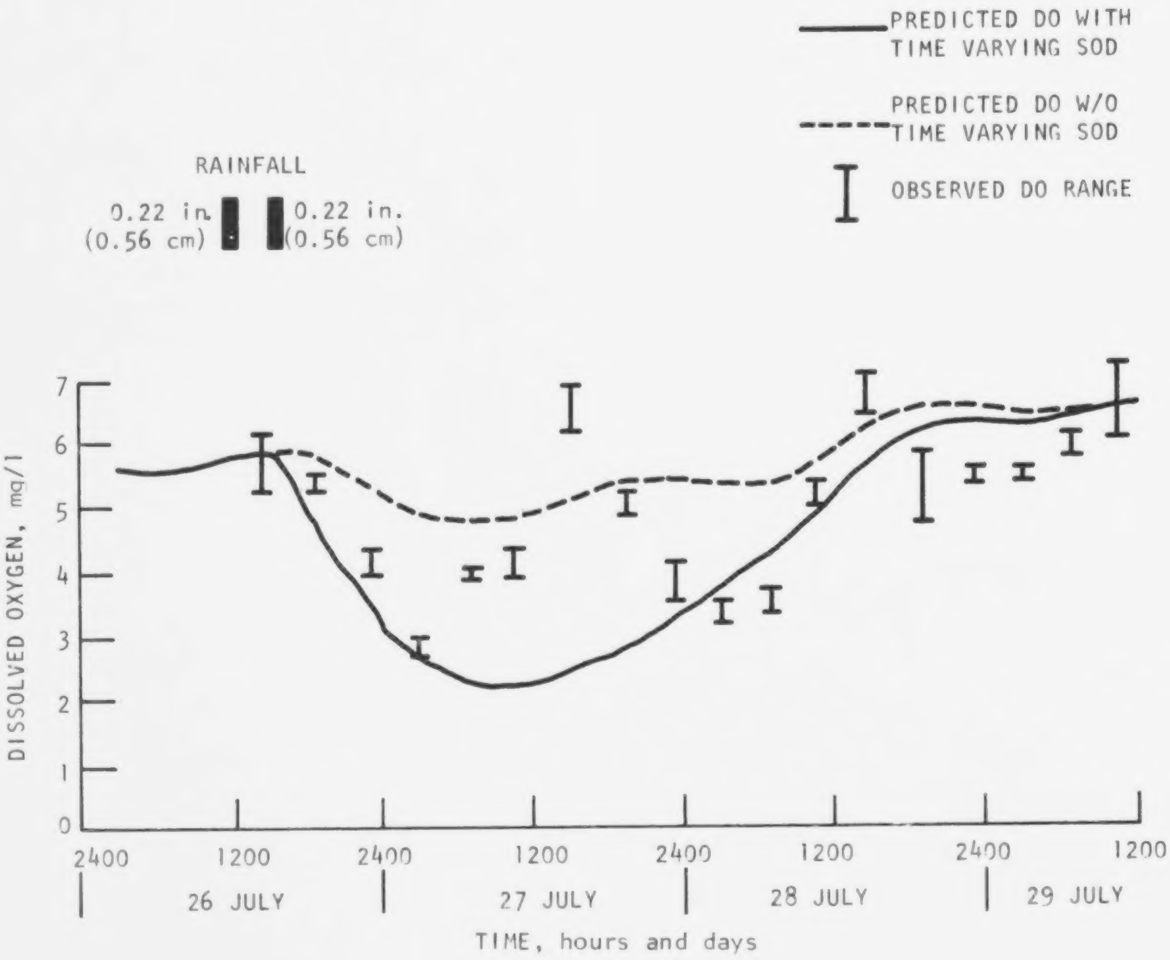

Figure A-14. Wet weather verification results for Wells Street, July 26-29, 1978 (Survey 6).

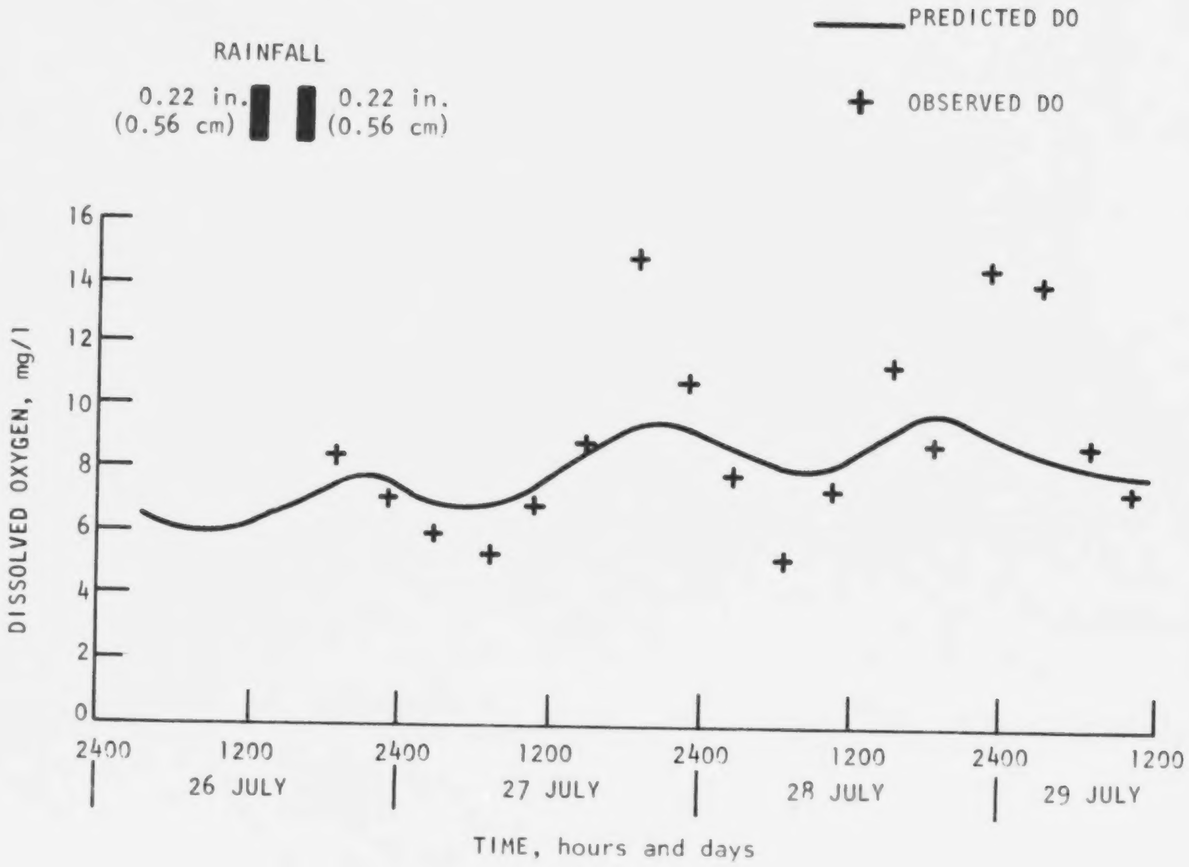

Figure A-15. Wet Weather verification results for North Avenue, July 26-29, 1978 (Survey 6).

APPENDIX TABLES
(A-1 through A-16)

TABLE A-1. SUMMARY OF MILWAUKEE RIVER INTENSIVE MONITORING RESULTS FOR THE DRY WEATHER SURVEY, SEPTEMBER 21-24, 1976 (SURVEY 1) (DAILY AVERAGES)

Date	Location	TS, mg/l	SS, mg/l	VSS, mg/l	BOD_5, mg/l	COD, mg/l	TOC, mg/l	NH_3-N, mg/l	NO_2 & NO_3-N, mg/l	Org.N, mg/l	Total P, mg/l	Fecal coliforms, MPN/100 ml
Sept. 21	Capitol Drive	-	-	-	2.5	-	-	0.03	0.02	-	0.12	2100
	North Ave	-	-	-	4.0	-	-	0.02	0.02	-	0.15	2470
	Humboldt Avenue	-	-	-	5.0	-	-	0.11	0.06	-	0.20	5200
	Walnut Street	-	-	-	3.0	-	-	0.34	0.09	-	0.19	7450
	St. Paul Avenue	-	-	-	3.0	-	-	0.58	0.17	-	0.12	4120
Sept. 22	Capitol Drive	-	-	-	3.0	-	-	0.05	0.02	-	0.13	507
	North Ave.	-	-	-	-	-	-	0.01	0.01	-	0.14	5630
	Humboldt Avenue	-	-	-	-	-	-	0.21	0.08	-	0.23	11300
	Walnut Street	-	-	-	-	-	-	0.36	0.09	-	0.20	13100
	St. Paul Avenue	-	-	-	-	-	-	0.62	0.22	-	0.13	708
Sept. 23	Capitol Drive	-	-	-	3.0	-	-	0.02	0.02	-	0.13	207
	North Ave.	-	-	-	-	-	-	0.04	0.02	-	0.16	252
	Humboldt Avenue	-	-	-	-	-	-	0.18	0.08	-	0.22	2000
	Walnut Street	-	-	-	-	-	-	0.38	0.10	-	0.20	10300
	St. Paul Avenue	-	-	-	-	-	-	0.71	0.26	-	0.14	210

TABLE A-2. SUMMARY OF MILWAUKEE RIVER INTENSIVE MONITORING RESULTS FOR DISSOLVED OXYGEN AND TEMPERATURE DURING DRY WEATHER, SEPTEMBER 21-24, 1976 (SURVEY 1).

		Dissolved oxygen - mg/l			Temperature	
Date	Site	Average	Minimum	Maximum	Minimum	Maximum
September 21	Capitol Drive	7.0	5.6	8.5	15.0	19.5
	North Ave. Dam	7.1	5.4	8.4	16.0	18.0
	Humboldt Avenue	6.9	6.0	7.8	16.0	19.0
	Walnut Street	4.9	3.5	6.3	16.0	20.0
	St. Paul Avenue	1.6	0.1	2.3	19.0	24.0
September 22	Capitol Drive	6.3	5.4	7.2	9.0	18.5
	North Ave. Dam	6.7	5.7	8.3	14.0	17.0
	Humboldt Avenue	5.9	4.8	7.8	16.5	18.5
	Walnut Street	4.4	3.2	5.6	17.8	19.5
	St. Paul Avenue	2.0	1.3	2.6	20.5	23.0
September 23	Capitol Drive	7.4	5.4	9.9	13.5	18.5
	North Ave. Dam	7.9	6.5	9.0	15.0	18.0
	Humboldt Avenue	6.3	5.0	8.2	16.0	18.5
	Walnut Street	5.4	4.1	6.6	16.0	19.5
	St. Paul Avenue	2.1	1.4	2.7	18.0	25.0

TABLE A-3. SUMMARY OF MILWAUKEE RIVER INTENSIVE MONITORING RESULTS FOR THE WET WEATHER SURVEY, MAY 31 - JUNE 3, 1977 (SURVEY 2)
(DAILY AVERAGES)

Date	Location	TS, mg/l	SS, mg/l	VSS, mg/l	BOD_5, mg/l	COD, mg/l	TOC, mg/l	NH_3-N, mg/l	NO_2 & NO_3-N, mg/l	Org.N, mg/l	Total P, mg/l	Fecal coliforms, MPN/100 ml
May 31	Port Washington Rd	572	46.8	14.3	3.3	38.3	22.0	0.09	0.04	1.87	0.29	503
	North Ave. Dam	543	35.8	31.7	4.7	38.8	21.0	0.35	0.09	1.19	0.32	6200
	Walnut Street	497	16.0	41.3	-	40.0	22.0	0.27	0.12	1.27	0.34	11225
	Wells Street	572	22.5	22.0	-	34.5	18.0	0.50	0.19	0.97	0.24	10450
	St. Paul Avenue	562	21.3	15.5	3.3	26.5	16.0	0.58	0.78	0.79	0.18	3195
June 1	Port Washington Rd	619	20.8	44.0	3.7	36.0	16.0	0.05	0.04	1.00	0.22	283
	North Ave. Dam	691	24.8	25.0	5.3	42.8	17.0	0.09	0.19	1.29	0.31	32300
	Walnut Street	606	17.0	12.5	-	41.5	16.0	0.47	0.13	0.96	0.26	4130
	Wells Street	518	10.8	8.0	-	33.5	14.0	0.41	0.14	1.15	0.20	8700
	St. Paul Avenue	441	26.0	16.0	3.7	24.3	10.0	0.80	0.19	0.87	0.14	4630
June 2	Port Washington Rd	1990	14.8	12.3	3.5	38.0	18.0	0.09	0.04	1.17	0.21	563
	North Ave. Dam	590	18.0	13.7	4.0	33.8	18.0	0.11	0.09	1.63	0.30	3090
	Walnut Street	591	12.8	10.7	-	33.3	16.0	0.30	1.14	1.45	0.24	2110
	Wells Street	497	13.0	10.7	-	27.0	12.0	0.42	0.10	1.52	0.24	5980
	St. Paul Avenue	440	12.3	8.0	4.0	21.3	10.0	0.66	0.12	1.12	0.28	9000
June 3	Port Washington Rd	1101	1.0	1.0	4.0	36.0	15.0	0.05	0.02	1.30	0.30	250
	North Ave. Dam	597	13.0	6.0	3.0	35.0	30.0	0.22	0.03	0.76	0.28	740
	Walnut Street	962	8.0	8.0	-	80.0	19.0	0.53	0.19	3.27	0.65	320
	Wells Street	505	17.0	2.0	-	25.0	13.0	0.70	0.13	0.85	0.27	270
	St. Paul Avenue	489	-	11.0	3.0	16.0	-	0.79	0.12	1.11	0.16	16000

TABLE A-4. SUMMARY OF MILWAUKEE RIVER INTENSIVE MONITORING RESULTS FOR DISSOLVED OXYGEN AND TEMPERATURE DURING WET WEATHER, MAY 31-JUNE 2, 1977 (SURVEY 2)

		Dissolved oxygen - mg/l			Temperature	
Date	Site	Average	Minimum	Maximum	Minimum	Maximum
May 31	Port Washington Rd.	7.8	6.8	9.4	20.0	23.0
	North Ave. Dam	3.3	1.4	5.0	20.0	21.8
	Walnut Street	3.6	2.3	4.3	20.0	22.0
	Wells Street	1.4	0.3	3.0	22.5	19.0
	St. Paul Avenue	2.5	1.6	3.5	18.0	21.5
June 1	Port Washington Rd.	6.3	5.2	7.2	20.2	21.5
	North Ave. Dam	3.8	3.4	4.0	20.0	21.0
	Walnut Street	2.7	2.1	3.9	21.0	22.4
	Wells Street	0.4	0.0	2.4	19.2	22.5
	St. Paul Avenue	2.4	0.9	6.2	16.2	22.0
June 2	Port Washington Rd.	8.6	5.2	11.0	19.8	22.0
	North Ave. Dam	5.3	2.9	6.8	19.5	21.5
	Walnut Street	3.2	2.1	4.3	20.0	22.2
	Wells Street	0.8	0.1	2.4	19.0	22.5
	St. Paul Avenue	1.9	0.6	3.6	16.0	23.0

TABLE A-5. SUMMARY OF MILWAUKEE RIVER INTENSIVE MONITORING RESULTS FOR THE WET WEATHER SURVEY, JUNE 18-20, 1977 (SURVEY 3) (DAILY AVERAGES)

Date	Location	TS, mg/l	SS, mg/l	VSS, mg/l	BOD_5, mg/l	COD, mg/l	TOC, mg/l	NH_3-N, mg/l	NO_2 & NO_3-N, mg/l	Org.N, mg/l	Total P, mg/l	Fecal coliforms, MPN/100 ml
June 18	Port Washington Rd	481	43.3	19.5	-	-	23.0	0.04	0.04	-	0.20	8780
	North Ave. Dam	477	40.3	19.5	-	-	22.0	0.01	0.02	-	0.20	8880
	Walnut Street	461	23.0	13.2	-	-	21.0	0.02	0.03	-	0.20	27200
	Wells Street	453	23.2	13.2	-	-	19.0	0.05	0.09	-	0.20	33200
	St. Paul Avenue	454	23.2	11.8	-	-	17.5	0.06	0.12	-	0.16	51800
June 19	Port Washington Rd	419	29.2	16.2	-	-	21.0	0.01	0.02	-	0.16	768
	North Ave. Dam	452	33.6	16.8	-	-	21.0	0.03	0.01	-	0.24	450
	Walnut Street	432	26.6	14.2	-	-	18.5	0.02	0.01	-	0.23	238
	Wells Street	446	20.6	12.4	-	-	19.0	0.11	0.02	-	0.17	1260
	St. Paul Avenue	406	18.4	8.6	-	-	17.0	0.21	0.08	-	0.16	11530
June 20	Port Washington Rd	518	19.0	10.8	-	-	-	0.02	0.01	-	0.14	-
	North Ave. Dam	444	25.8	13.2	-	-	-	0.02	0.02	-	0.20	-
	Walnut Street	423	26.2	11.0	-	-	-	0.10	0.02	-	0.11	-
	Wells Street	390	14.4	7.6	-	-	-	0.23	0.03	-	0.12	-
	St. Paul Avenue	353	12.8	5.8	-	-	-	0.28	0.06	-	0.10	-

TABLE A-6. SUMMARY OF THE MILWAUKEE RIVER INTENSIVE MONITORING RESULTS FOR DISSOLVED OXYGEN AND TEMPERATURE DURING WET WEATHER, JUNE 18-20, 1977 (SURVEY 3)

		Dissolved oxygen - mg/l			Temperature	
Date	Site	Average	Minimum	Maximum	Minimum	Maximum
June 18	Port Washington Rd.	11.3	9.4	12.5	22.5	24.5
	North Ave. Dam	8.8	5.5	13.4	22.0	24.5
	Walnut Street	7.3	3.1	9.8	21.8	23.5
	Wells Street	5.1	3.6	7.2	21.0	22.5
	St. Paul Avenue	3.5	0.2	6.6	19.0	23.2
June 19	Port Washington Rd.	12.2	10.9	13.3	24.0	25.2
	North Ave. Dam	9.3	4.6	13.3	23.8	25.5
	Walnut Street	6.6	3.8	8.8	23.5	25.8
	Wells Street	3.6	1.6	6.0	23.5	25.2
	St. Paul Avenue	2.9	0.9	4.6	20.0	24.0
June 20	Port Washington Rd.	9.3	4.6	13.3	24.0	25.2
	North Ave. Dam	6.7	5.7	7.8	22.0	25.2
	Walnut Street	5.0	1.8	6.7	23.5	26.0
	Wells Street	2.7	1.6	4.8	20.0	26.2
	St. Paul Avenue	3.6	2.1	3.8	16.0	23.8

TABLE A-7. SUMMARY OF MILWAUKEE RIVER INTENSIVE MONITORING RESULTS FOR THE WET WEATHER SURVEY, AUGUST 4-8, 1977 (SURVEY 4) (DAILY AVERAGES)

Date	Location	TS, mg/l	SS, mg/l	VSS, mg/l	BOD_5, mg/l	COD, mg/l	TOC, mg/l	NH_3-N, mg/l	NO_2 & NO_3-N, mg/l	Org.N, mg/l	Total P, mg/l	Fecal coliforms, NPN/100 ml
Aug. 4	Port Washington Rd	490	26.5	11.5	-	-	22.3	0.10	-	-	0.23	115000
	North Ave. Dam	513	32.3	13.8	-	-	26.8	0.24	-	-	0.36	61000
	Walnut Street	470	27.0	10.5	-	-	21.3	0.29	-	-	0.33	46100
	Wells Street	438	37.3	11.3	-	-	26.3	0.10	-	-	0.30	25000
	St. Paul Avenue	434	34.7	13.3	-	-	21.7	0.22	-	-	0.26	186000
Aug. 5	Port Washington Rd	466	27.3	12.0	-	-	24.8	0.11	-	-	0.30	21900
	North Ave. Dam	493	35.0	13.5	-	-	21.3	0.16	-	-	0.31	22400
	Walnut Street	478	26.8	10.0	-	-	21.0	0.24	-	-	0.35	41000
	Wells Street	469	29.0	15.8	-	-	25.8	0.10	-	-	0.45	119000
	St. Paul Avenue	429	23.5	13.3	-	-	23.8	0.10	-	-	0.40	500000
Aug. 6	Port Washington Rd	435	35.0	12.3	-	-	18.3	0.12	-	-	0.26	1310
	North Ave. Dam	444	41.0	11.5	-	-	23.0	0.19	-	-	0.30	1910
	Walnut Street	440	34.8	12.5	-	-	21.5	0.20	-	-	0.28	4700
	Wells Street	431	21.0	9.0	-	-	20.3	0.22	-	-	0.25	192000
	St. Paul Avenue	405	16.3	8.8	-	-	20.5	0.19	-	-	0.32	250000

(continued)

TABLE A-7 (continued).

Date	Location	TS, mg/1	SS, mg/1	VSS, mg/1	BOD_5, mg/1	COD, mg/1	TOC, mg/1	NO_3-N, mg/1	NO_2 & NO_3-N, mg/1	Org.N, mg/1	Total P, mg/1	Fecal coliforms, MPN/100 ml
Aug. 7	Port Washington Rd	523	39.3	17.8	-	-	22.3	0.10	-	-	0.24	295
	North Ave. Dam	541	43.5	16.0	-	-	23.0	0.10	-	-	0.29	405
	Walnut Street	560	42.0	15.8	-	-	22.5	0.10	-	-	0.29	645
	Wells Street	514	33.3	12.5	-	-	21.0	0.18	-	-	0.26	175
	St. Paul Avenue	515	30.3	11.3	-	-	21.0	0.26	-	-	0.24	4800
Aug. 8	Port Washington Rd	515	44.0	17.0	-	-	22.5	0.01	-	-	0.24	170
	North Ave. Dam	500	47.5	13.5	-	-	26.5	0.01	-	-	0.22	360
	Walnut Street	460	45.5	19.5	-	-	24.5	0.01	-	-	0.27	900
	St. Paul Avenue	487	29.5	13.0	-	-	22.5	0.23	-	-	0.20	4800

TABLE A-8. SUMMARY OF THE MILWAUKEE RIVER INTENSIVE MONITORING RESULTS FOR DISSOLVED OXYGEN AND TEMPERATURE DURING WET WEATHER, AUGUST 4-8, 1977 (SURVEY 4)

		Dissolved oxygen - mg/l			Temperature	
Date	Site	Average	Minimum	Maximum	Minimum	Maximum
August 4	Port Washington Rd.	6.8	5.6	8.4	21.5	23.0
	North Ave. Dam	4.0	2.1	5.2	22.2	23.0
	Walnut Street	-	-	-	19.8	23.0
	Wells Street	1.2	0.1	2.9	19.5	21.5
	St. Paul Avenue	1.0	0.0	2.8	20.0	21.5
August 5	Port Washington Rd.	6.2	5.4	8.5	21.5	25.0
	North Ave. Dam	5.5	3.4	8.6	21.9	23.5
	Walnut Street	5.3	3.3	7.4	21.5	23.5
	Wells Street	0.6	0.0	3.2	21.5	23.5
	St. Paul Avenue	0.3	0.0	1.4	20.0	23.5
August 6	Port Washington Rd	9.3	5.5	15.0	21.0	24.0
	North Ave. Dam	7.6	4.9	11.4	21.5	23.5
	Walnut Street	6.6	4.0	12.0	21.0	24.0
	Wells Street	3.2	0.4	7.7	22.0	24.0
	St. Paul Avenue	1.6	0.0	5.4	19.5	23.5
August 7	Port Washington Rd.	9.5	8.0	11.4	22.5	24.0
	North Ave. Dam	8.3	5.8	11.0	22.0	25.0
	Walnut Street	7.7	6.2	9.3	23.0	24.5
	Wells Street	6.0	4.7	7.5	23.0	24.0
	St. Paul Avenue	4.0	1.8	6.2	20.0	23.5

(continued)

TABLE A-8. (continued)

Date	Site	Dissolved oxygen - mg/l			Temperature	
		Average	Minimum	Maximum	Minimum	Maximum
August 8	Port Washington Rd.	8.6	7.5	9.4	22.0	24.0
	North Ave. Dam	7.1	5.4	9.0	22.0	24.0
	Walnut Street	8.0	7.3	8.8	22.0	24.0
	Wells Street	7.1	6.0	8.0	23.0	23.5
	St. Paul Avenue	5.2	3.8	6.2	18.0	23.0

TABLE A-9. SUMMARY OF MILWAUKEE RIVER INTENSIVE MONITORING RESULTS FOR THE WET WEATHER SURVEY, JUNE 16 - JUNE 18, 1978 (SURVEY 5)
(DAILY AVERAGES)

Date	Location	TS, mg/l	SS, mg/l	VSS, mg/l	BOD_5, mg/l	COD, mg/l	TOC, mg/l	NH_3-N, mg/l	NO_2 & NO_3-N, mg/l	Org.N, mg/l	Total P, mg/l	Fecal coliforms, MPN/100 ml
June 16	Port Washington Rd	-	-	-	3	32	30	<.10	.35	-	.14	1930
	North Ave. Dam	-	-	-	7	37	38	.13	.22	-	.18	3000
	Walnut Street	-	-	-	8	38	40	.17	.15	-	.18	10200
	Wells Street	-	-	-	11	45	42	.46	.14	-	.23	21500
	St. Paul Avenue	-	-	-	4	41	37	.67	.15	-	.24	13300
June 17	Port Washington Rd	-	-	-	-	39	13	.18	.58	-	.12	12100
	North Ave. Dam	-	-	-	-	39	11	.22	.51	-	.22	18800
	Walnut Street	-	-	-	-	37	11	.29	.54	-	.20	64000
	Wells Street	-	-	-	-	53	19	.35	.50	-	.37	94700
	St. Paul Avenue	-	-	-	-	44	14	.29	.50	-	.23	55700
June 18	Port Washington Rd	-	-	-	8	39	12	.23	1.73	-	.20	9900
	North Ave. Dam	-	-	-	10	39	12	.26	1.56	-	.25	11000
	Walnut Street	-	-	-	9	37	21	.29	1.51	-	.20	9800
	Wells Street	-	-	-	9	41	12	.39	1.42	-	.24	50800
	St. Paul Avenue	-	-	-	10	41	13	.34	1.37	-	.27	13800

Table A-10. SUMMARY OF THE MILWAUKEE RIVER INTENSIVE MONITORING RESULTS FOR DISSOLVED OXYGEN AND TEMPERATURE DURING WET WEATHER, JUNE 16-19, 1978 (SURVEY 5)

Date	Site	Dissolved oxygen, mg/l			Temperature	
		Ave.	Min.	Max.	Min.	Max.
6/16/78	Port Washington Rd.	7.8	6.6	8.1	18.0	20.5
	North Ave. Dam	7.5	6.0	9.8	17.0	20.5
	Walnut St.	8.0	6.8	9.4	17.5	21.0
	Wells St.	6.0	5.1	7.2	17.5	21.0
	St. Paul Ave.	3.9	2.2	5.6	18.0	20.75
6/17/78	Port Washington Rd.	6.3	6.1	8.8	18.0	20.5
	North Ave. Dam	7.2	6.2	8.2	18.0	20.0
	Walnut St.	7.5	5.2	8.5	18.0	20.5
	Wells St.	6.7	6.2	7.7	17.5	20.0
	St. Paul Ave.	5.7	3.7	7.0	17.5	20.0
6/18/78	Port Washington Rd.	6.7	4.1	8.3	18.5	22.0
	North Ave. Dam	6.8	4.6	8.5	18.25	21.0
	Walnut Street	7.7	4.6	9.1	18.5	22.0
	Wells St.	7.3	4.3	8.1	19.0	21.0
	St. Paul Ave.	6.3	4.2	7.7	19.0	21.0
6/19/78	Port Washington Rs.	7.6	7.6	7.6	19.5	19.5
	North Ave. Dam	7.2	7.2	7.2	19.5	21.0
	Walnut St.	8.2	8.0	8.5	19.5	21.0
	Wells St.	7.6	7.5	7.9	20.0	21.0
	St. Paul Ave.	7.1	7.0	7.5	20.0	21.0

TABLE A-11. SUMMARY OF MILWAUKEE RIVER INTENSIVE MONITORING RESULTS FOR THE WET WEATHER SURVEY, JULY 26 - JULY 29, 1978 (SURVEY 6)
(DAILY AVERAGES)

Date	Location	TS, mg/1	SS, mg/1	VSS, mg/1	BOD_5, mg/1	COD, mg/1	TOC, mg/1	NH_3-N, mg/1	NO_2 & NO_3-N, mg/1	Org.N, mg/1	Total P, mg/1	Fecal coliforms, MPN/100 ml
July 26	Port Washington Rd	-	-	-	10	45	16.5	.17	.06	-	.15	680
	North Ave. Dam	-	-	-	5	49	15.5	.22	.08	-	.24	28000
	Walnut Street	-	-	-	7	47	16.5	.22	.12	-	.23	36700
	Wells Street	-	-	-	14	56	20.5	.44	.12	-	.24	26700
	St. Paul Avenue	-	-	-	10	51	19.0	.57	.06	-	.22	21700
July 27	Port Washington Rd	-	-	-	6	44	14.5	.25	.15	-	.15	6700
	North Ave. Dam	-	-	-	7	46	15.0	.19	.12	-	.15	13800
	Walnut Street	-	-	-	8	46	15.5	.38	.10	-	.20	15900
	Wells Street	-	-	-	8	42	14.0	.54	.12	-	.18	50200
	St. Paul Avenue	-	-	-	8	44	14.8	.35	.12	-	.23	38300
July 28	Port Washington Rd	-	-	-	7	47	16.5	.16	.12	-	.15	680
	North Ave. Dam	-	-	-	8	47	15.3	.20	.06	-	.19	12500
	Walnut Street	-	-	-	9	44	14.8	.09	.06	-	.17	3540
	Wells Street	-	-	-	8	39	14.3	.23	.10	-	.18	5870
	St. Paul Avenue	-	-	-	9	34	14.0	.32	.08	-	.19	33300
July 29	Port Washington Rd	-	-	-	5	41	17.0	.25	<.01	-	.14	310
	North Avenue Dam	-	-	-	6	47	18.0	.24	.05	-	.23	9000
	Walnut Street	-	-	-	7	42	16.0	.04	<.01	-	.18	1400
	Wells Street	-	-	-	5	41	17.0	.21	.05	-	.19	1300
	St. Paul Avenue	-	-	-	6	34	13.5	.51	.17	-	.16	30000

TABLE A-12. SUMMARY OF THE MILWAUKEE RIVER INTENSIVE MONITORING RESULTS FOR DISSOLVED OXYGEN AND TEMPERATURE DURING WET WEATHER, JULY 26-29, 1978 (SURVEY 6)

		Dissolved oxygen, mg/l			Temperature	
Date	Site	Ave.	Min.	Max.	Min.	Max.
7/26/78	Port Washington Rd.	9.2	8.8	9.8	23.0	25.0
	North Ave. Dam	7.7	7.2	8.6	23.0	23.5
	Walnut St.	7.5	5.6	8.6	21.0	23.0
	Wells St.	5.3	4.0	6.3	21.0	24.0
	St. Paul Ave.	4.1	2.4	5.8	22.5	25.0
7/27/78	Port Washington Rd.	10.3	7.0	15	22.0	25.0
	North Ave. Dam	9.0	5.6	15	22.0	25.0
	Walnut St.	6.4	4.5	9.5	21.0	24.0
	Wells St.	4.4	2.8	7.0	21.0	24.0
	St. Paul Ave.	2.7	0.0	5.9	21.0	23.5
7/28/78	Port Washington Rd.	12.1	8.2	15	22.0	23.0
	North Ave. Dam	9.3	5.4	14.6	21.0	23.0
	Walnut St.	7.4	3.7	12.2	21.5	24.0
	Wells St.	5.0	3.2	7.2	22.5	24.0
	St. Paul Ave.	3.4	0.7	6.0	21.0	26.0
7/29/78	Port Washington Rd.	13.3	12.1	15	22.5	23.0
	North Ave. Dam	10.3	7.7	14.1	22.0	23.0
	Walnut St.	8.1	7.4	10.2	22.0	23.0
	Wells St.	6.1	5.4	7.3	22.5	24.0
	St. Paul Ave.	3.6	2.7	5.6	18.5	25.0

TABLE A-13. ESTABROOK PARK RIVER FLOW DURING THE INTENSIVE MONITORING SURVEYS OF THE MILWAUKEE RIVER

Survey	Date	Average flow, cfs	Maximum flow, cfs	Minimum flow, cfs
1	9/21/76	79	85	77
	9/22/76	73	77	72
	9/23/76	70	72	68
2	5/31/77	89	154	56
	6/1/77	74	80	70
	6/2/77	71	80	64
	6/3/77	86	87	77
3	6/18/77	250	393	181
	9/19/77	202	215	181
	6/20/77	188	205	163
4	8/4/77	170	418	98
	8/5/77	344	405	176
	8/6/77	461	498	312
	8/7/77	448	491	399
	8/8/77	383	498	334
5	6/16/78	453	915	206
	6/17/78	1623	3112	372
	6/18/78	1645	2302	933
6	7/26/78	342	568	275
	7/27/78	268	306	240
	7/28/78	247	255	240
	7/29/78	255	255	255

NOTE: m^3/sec = 0.028 × cfs

TABLE A-14. SUMMARY OF MILWAUKEE RIVER CHLOROPHYLL _a_ CONCENTRATIONS.

Location	Chlorophyll _a_, mg/m^3							
	June 24	July 8	July 15	July 20	July 28	August 8	August 30	September 27
Brown Deer	64.97	24.55	69.5	152.0	--	22.75	144.05	16.31
Silver Spring	37.85	54.22	68.7	132.0	--	14.92	--	20.19
Port Washington	33.08	38.61	38.7	125.3	--	25.24	88.85	13.36
North Avenue	--	38.00	75.3	121.3	26.86	15.16	--	19.71
Walnut	--	23.49	81.3	48.7	26.84	20.99	61.00	19.46
Wells	--	19.71	59.4	35.0	28.86	--	--	16.40
St. Paul	16.99	15.17	22.0	15.9	23.90	12.36	16.50	17.93
Inner Harbor	--	--	--	--	--	--	17.13	--
Outer Harbor	--	--	--	--	--	--	3.48	--

TABLE A-15. RESULTS OF CHEMICAL ANALYSIS OF MILWAUKEE RIVER SEDIMENTS SAMPLED IN MAY AND JUNE, 1977

	Location						
	River Junction			Marine Bank			
Parameter	East	Mid-Top	West	East	Mid-Top	Mid-Bottom	West
Total Solids - %	-	57.6	26.9	17.3	14.9	55.5	14.0
Volatile Total Solids - %	-	3.0	2.9	2.0	2.1	3.6	1.9
COD-mg/kg	-	156,000	118,200	264,000	167,400	86,000	120,800
Nitrite+Nitrate Nitrogen-mg/kg	-	0.12	0.06	0.05	0.37	0.12	0.13
Ammonia-Nitrogen-mg/kg	-	77	420	197	271	60	695
Total Phosphorus-mg/kg	-	616	1,648	1,620	1,493	475	1,706
Iron-mg/kg	-	21,500	20,900	21,600	16,400	11,700	26,300
Cadmium-mg/kg	-	23	15	10	9	10	30
Zinc-mg/kg	-	53	443	500	296	63	758
Lead-mg/kg	-	34	628	568	6,350	63	1,020
Copper-mg/kg	-	25	117	128	70	21	263
pH	-	8.0	7.4	7.3	6.6	7.9	7.5

(continued)

TABLE A-15 (continued).

	Location							
	Highland Blvd				Walnut Street			
Parameter	East	Mid-Top	Mid-Bottom	West	East	Mid-Top	Mid-Bottom	West
Total Solids - %	18.0	37.8	33.2	30.4	36.4	40.8	36.9	27.8
Volatile Total Solids-%	2.2	4.1	4.8	3.6	3.4	4.1	5.1	3.1
COD-mg/kg	236,000	159,000	380,000	151,200	127,000	127,800	243,000	185,100
Nitrite+Nitrate Nitrogen-mg/kg	0.05	0.04	0.04	0.05	0.85	0.04	0.03	0.03
Ammonia-Nitrogen-mg/kg	302	390	743	378	440	394	572	451
Total Phosphorus-mg/kg	1,510	1,630	2,840	1,059	1,510	1,728	2,660	1,144
Iron-mg/kg	19,900	20,900	19,800	21,200	16,500	19,800	20,800	23,300
Cadmium-mg/kg	13	13	23	19	11	18	16	37
Zinc-mg/kg	459	495	783	493	433	554	826	749
Lead-mg/kg	574	434	623	587	824	471	703	582
Copper-mg/kg	117	107	222	163	122	528	198	112
pH	7.4	6.7	6.7	7.4	7.3	7.4	7.0	6.7

(continued)

TABLE A-15 (continued).

	Location					
	Humboldt Ave.			North Ave.		
Parameter	East	Mid	West	East	Mid	West
Total Solids - %	9.8	31.4	26.7	38.4	38.4	20.8
Volatile Total Solids - %	2.0	3.7	2.0	3.3	2.8	1.9
COD-mg/kg	293,800	122,700	129,000	119,000	112,500	195,000
Nitrite+Nitrate Nitrogen-mg/kg	0.10	0.03	0.03	0.05	0.04	0.07
Ammonia-Nitrogen-mg/kg	388	241	188	168	332	22
Total Phosphorus-mg/kg	1,580	862	830	1,147	1,230	2,000
Iron-mg/kg	19,300	12,900	11,000	15,200	19,700	25,000
Cadmium-mg/kg	13	16	9	9	14	12
Zinc-mg/kg	556	314	259	365	357	492
Lead-mg/kg	643	563	321	505	396	648
Copper-mg/kg	118	92	64	93	78	141
pH	6.7	7.4	7.5	7.0	7.2	7.0

(continued)

TABLE A-15 (continued).

	Location					
	Burleigh St.			Hubbard Park		
Parameter	East	Mid	West	East	Mid	West
Total Solids - %	33.9	51.6	21.2	40.4	28.9	78.6
Volatile Total Solids - %	3.5	1.2	2.2	3.3	2.9	2.0
COD-mg/kg	150,000	41,770	166,000	148,600	148,400	8,820
Nitrite+Nitrate Nitrogen-mg/kg	0.02	0.03	0.09	0.06	0.03	0.01
Ammonia-Nitrogen-mg/kg	163	130	334	163	210	20
Total Phosphorus-mg/kg	1,243	407	1,504	1,476	1,708	69
Iron-mg/kg	19,000	7,00	19,000	16,300	22,800	6,800
Cadmium-mg/kg	9	16	10	8	14	3
Zinc-mg/kg	438	110	432	425	393	137
Lead-mg/kg	530	186	668	716	457	49
Copper-mg/kg	3	44	107	123	89	125
pH	6.7	7.5	7.3	6.9	7.4	7.6

(continued)

TABLE A-15 (continued).

	Location					
	Lincoln Park- South			Lincoln Park-Central		
Parameter	East	Mid	West	East	Mid	West
Total Solids - %	30.7	35.4	62.7	81.7	78.8	80.7
Volatile Total Solids - %	0.4	0.4	1.8	0.8	0.4	1.5
COD-mg/kg	3,390	2,160	33,600	2,510	577	14,800
Nitrite+Nitrate Nitrogen-mg/kg	0.05	2.12	0.02	<0.01	<0.01	0.02
Ammonia-Nitrogen-mg/kg	14	9	33	10	10	17
Total Phosphorus-mg/kg	32	28	248	26	18	147
Iron-mg/kg	34,000	9,700	15,500	5,300	4,770	6,600
Cadmium-mg/kg	7	6	4	2	5	3
Zinc-mg/kg	90	86	171	37	29	50
Lead-mg/kg	67	26	239	40	38	58
Copper-mg/kg	32	10	36	5	5	9
pH	5.7	8.0	7.0	7.1	7.9	6.9

TABLE A-15 (continued).

Parameter	Location Lincoln Park-North East	 Mid	 West	 Dean Rd. East	 Mid	 West
Total Solids - %	77.4	41.7	85.5	76.3	61.7	82.4
Volatile Total Solids - %	1.0	1.2	1.3	1.0	0.4	1.2
COD-mg/kg	4,370	35,200	8,940	11,800	2,000	2,800
Nitrite+Nitrate Nitrogen-mg/kg	0.01	0.02	0.29	0.01	0.01	0.01
Ammonia-Nitrogen-mg/kg	21	34	9	85	41	34
Total Phosphorus-mg/kg	40	290	72	133	23	42
Iron-mg/kg	5,100	17,600	4,300	5,570	5,360	6,590
Cadmium-mg/kg	2	5	3	7	8	4
Zinc-mg/kg	28	48	24	42	21	31
Lead-mg/kg	25	27	29	33	24	33
Copper-mg/kg	5	16	5	2	2	6
pH	7.7	7.7	7.3	7.5	7.2	8.1

TABLE A-16. LABORATORY ANALYSIS RESULTS - SEDIMENT SAMPLES OF JANUARY 21, 1976

Parameter	Units	Harbor	St. Paul	Wells	Cherry	State	Walnut	Holton
pH		6.6	6.7	7.1	6.7	6.9	6.9	6.9
Oxidation-reduction potential	millivolts	-207	-237	-238	-216	-220	-256	-230
Total solids	%	16.7	29.1	22.5	20.0	24.2	58.4	34.0
Volatile solids	% (dry wt)	19.9	11.8	12.7	15.4	12.6	5.5	9.7
BOD_{20}	mg/kg (dry wt)	47,900	16,100	28,400	21,500	18,600	7.000	20,000
COD	mg/kg (dry wt)	260,800	160,000	180,400	220,200	185,300	117,600	152,000
TOC	mg/kg (dry wt)	80,800	61,900	53,300	70,000	66,100	22,230	64,700
Ammonia-N	mg/kg (dry wt)	4,200	269	321	620	432	68	590
Nitrate-N	mg/kg (dry wt)	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
Nitrite-N	mg/kg (dry wt)	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
Total phosphorus	mg/kg (dry wt)	6,470	1,560	2,110	1,940	1,870	616	1,530
Iron	mg/kg (dry wt)	24,000	18,000	21,000	18,000	21,000	8,200	18,000
Cadmium	mg/kg (dry wt)	39	10	11	10	11	6.1	11
Lead	mg/kg (dry wt)	285	405	600	535	465	310	685
Copper	mg/kg (dry wt)	165	111	122	102	121	73	112
Zinc	mg/kg (dry wt)	705	410	470	465	470	255	455